Technical Report CHL-97-5
February 1999

US Army Corps
of Engineers
Waterways Experiment
Station

Monitoring Completed Navigation Projects Program

Burns Harbor, Indiana, Monitoring Study

Volume II: Detailed Approach and Results

Technical Editors David McGehee, Terri Prickett, WES
Heidi Moritz, Chicago District

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Prepared for Headquarters, U.S. Army Corps of Engineers

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Final report

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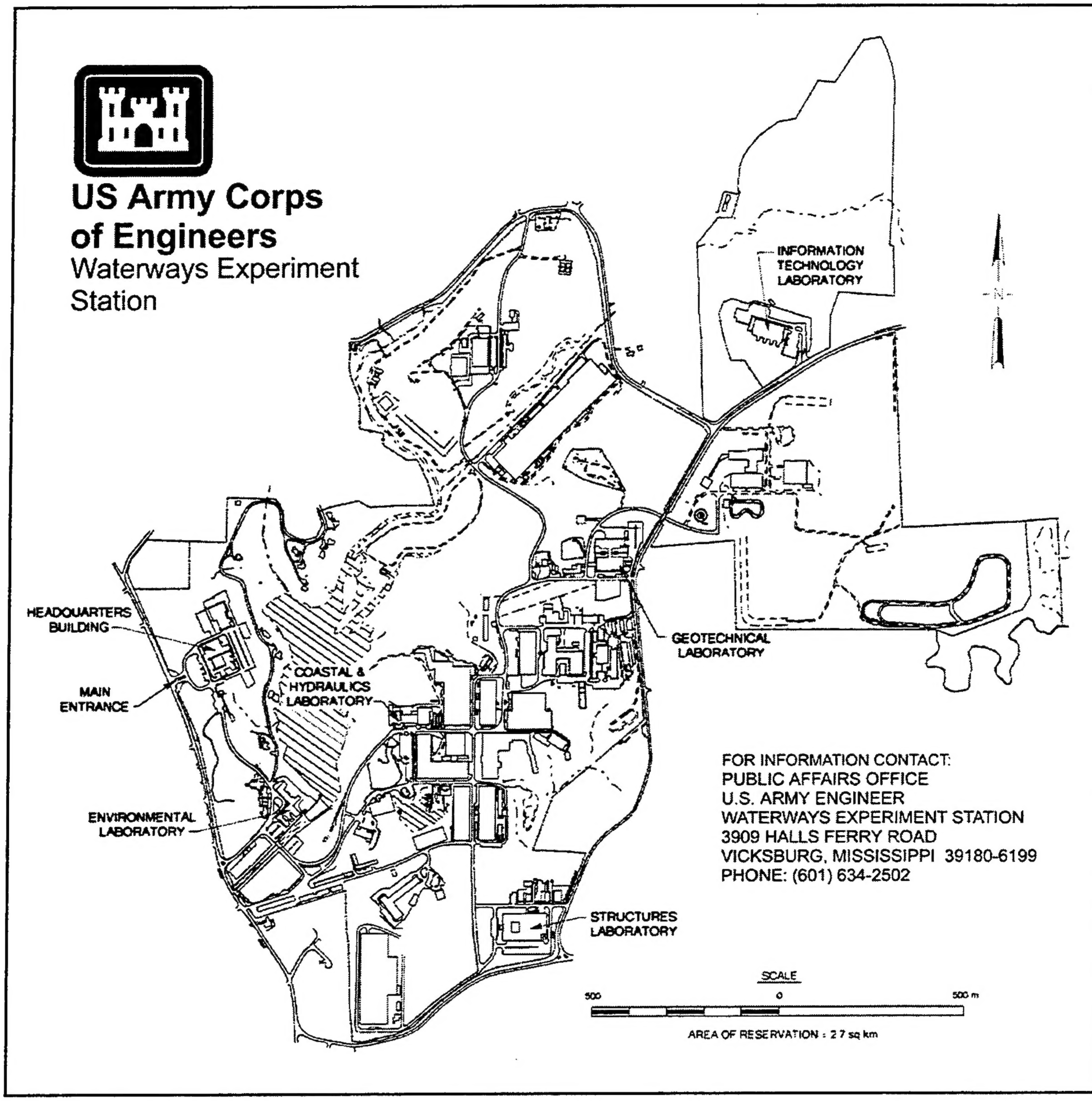

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Preface

This report is being published by the U.S. Army Engineer Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory (CHL). The CHL was formed in October 1996 with the merger of the WES Coastal Engineering Research Center (CERC) and Hydraulics Laboratory (HL). Dr. Robert W. Whalin was Director of WES at the time of this report publication. Dr. James R. Houston is the Director of the CHL and Mr. Charles C. Calhoun, Jr., is Assistant Director.

This report was prepared by CHL, and is a product of the Monitoring Completed Coastal Projects Program (MCCP), which was renamed in October 1996 as the Monitoring Completed Navigation Projects (MCNP) Program. It represents a joint effort between CHL and the U.S. Army Engineer District, Chicago (NCC). The MCNP Program Manager when the study was initiated was Mr. J. Michael Hemsley. The Program Manager at the conclusion of the study was Ms. Carolyn Holmes. Technical Monitors of the MCNP Program are Messrs. John H. Lockhart, Jr., Charles B. Chesnutt, and Barry W. Holliday.

The Principal Investigator of the Burns Harbor work unit was Mr. David McGehee of CHL. During the course of this study he was supervised by Mr. William Preslan, Chief, Prototype Measurement and Analysis Branch (PMAB), and Mr. Thomas Richardson, Chief, Engineering Development Division (EDD).

The NCC Principal Investigator during the course of this study was Ms. Heidi Moritz. Ms.Moritz was supervised by Mr. Harwood Herlocker and Mr. Utpal Bhattacharya, Chief, Geotechnical and Coastal Branch, and Mr. Joseph Jacobazzi, Chief, Engineering Division. She received considerable input and assistance from Mr. Hans Moritz, Ms. Joanne Milo, Mr. John Fornek, and Mr. Erik Matthews. NCC personnel were supervised by Mr. Joseph Jacobazzi. During the earlier phase of the study, NCC was ably represented by Messrs. Harry Krampitz and John Panganiban. Additionally, Mr. Charlie Johnson of the North Central Division provided valuable historical perspective and continuity through his familiarity with the project.

This report is printed in two volumes. Volume I provides an overview of the monitoring effort, including summaries of elements described in Volume II as well as additional analyses, results, and conclusions. It represents a collaboration between the two principal investigators. Additional analyses and summarization of the extensive "structural" sections were conducted by Ms. Terri Prickett of PMAB and Ms. Janean Shirley of the Information Technology Laboratory (ITL). Ms. Prickett was supervised by Mr. Preslan (PMAB), and Mr. Richardson (EDD). Ms. Shirley was supervised by Ms. Jamie Leach, Chief, Publishing Group, Mr. Bobby Baylot, Chief, Visual Production Center, and Dr. N. Radhakrishnan, Director, ITL.

Volume II contains independently prepared chapters with detailed descriptions of five major elements of the overall study, as outlined below. Technical Editors of Volume II were Mr. David McGehee, and Mses. Prickett and Moritz.

Chapter 1: "Project History" was written by Mses. Prickett and Moritz.

Chapter 2: "Results of Analysis of Wave Measurements at Burns Harbor" was written by Mr. McGehee and Dr. Joon Rhee, PMAB. Dr. Rhee was supervised by Mr. Preslan (PMAB), and Mr. Richardson (EDD).

Chapter 3: "Extremal Analysis of Burns Harbor Hindcast and Measured Wave Data" was written by Dr. Michael Andrew of Jackson, MS. Dr. Andrew is a private consultant and former CERC employee.

Chapter 4: "Evaluation of Breakwater Settlement" was written by Mr. John Andersen, and was supervised by Mr. W. Milton Myers, Chief, Soil Mechanics Branch (GS-S), Mr. Don C. Banks, Chief, Soil and Rock Mechanics Division (GS), and Dr. William Marcuson III, Director, Geotechnical Laboratory (GL).

Chapter 5: "Structural Stability Analysis" was written by Ms. Heidi Moritz and Mr. Hans Moritz of the USACE District, Chicago. Mr. Moritz was supervised by Mr. Harwood Herlocker and Mr. Utpal Bhattacharya, Chief, Geotechnical and Coastal Branch, and Mr. Joseph Jacobazzi, Chief, Engineering Division.

Organization and preparation of these reports were coordinated by Ms. Prickett of PMAB. She received assistance from Mses. Lula Davenport, Joy Brogdon, and Kathy Moore. Other PMAB personnel that provided valuable and patient assistance are Ms. Linda Lillycrop, Ms. Wendy Thompson, and Ms. Rhonda Lofton. Special appreciation is extended to Dr. Joon Rhee for his illumination of wave theories and Mr. Pat McKinney for his wizardry in programming.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic yards	0.764559	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	2.54	centimeters
miles (U.S. nautical)	1.852	kilometers
miles (U.S. statute)	1.609347	kilometers
pounds (mass) per cubic foot	16.02	kilograms per cubic meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
square feet	0.09290304	square meters
tons (force) per square foot	95.76052	kilopascals
tons (short)	0.9078	tons (metric)

1 Project History

by Terri L. Prickett[1] and Heidi P. Moritz[2]

Introduction

The following text describes the history of the Burns Harbor breakwater, including its design planning and authorization, model studies, construction, environmental loading, damage and maintenance, and subsequent studies. Breakwater design studies were conducted and conferences held to determine the most effective parameters for the design prior to its construction. See Chapter 5, Figure 5-1 in this volume for an illustration of the final layout of Burns Harbor. During post-construction years, the breakwater experienced significant damage. Additional studies were conducted in later years to determine if environmental loading and/or structural design contributed to the damage.

Project Authorization

Construction of the Burns Waterway Harbor was initially proposed to the Office of the Chief of Engineers (OCE) in 1931 to address needs for industrial expansion in northern Indiana. This proposal was considered unfavorable because anticipated benefits were limited. By August 1950, however, the U.S. Army Engineer District, Chicago (NCC) submitted a report to OCE that recognized the need for a commercial harbor in the northern Indiana area. OCE authorized preparation of a survey report for the harbor in February 1951.

NCC conducted the study of Burns Harbor and submitted the Great Lakes Harbor Study Interim Report to OCE on 16 February 1962. NCC's plan studied several alternative breakwater structures that included a laid-up placement, high-core breakwater alternative and a cellular steel sheetpile alternative.

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[2] Hydraulic Engineer, U.S. Army Engineer District, Chicago, IL.

In response to resolutions by the Committees on Public Works, United States Senate and House of Representatives, NCC submitted a favorable report to OCE on Burns Waterway Harbor in June, 1962. The Burns Waterway Harbor Project was authorized by the River and Harbor Act of 1965 (Public Law 8-9-298, 89th Congress). This act also stated that the Secretary of the Army would reimburse the State of Indiana for expenditure of funds used to construct such portions of the project as were approved by OCE and constructed under supervision of OCE.

Breakwater Design Sequence

Design of the harbor and rubble-mound breakwater was an iterative process involving several levels of review and revision. NCC initially proposed a breakwater design with a 30-ft- (9.1-m-)[1] deep entrance channel, a 27-ft- (8.2-m-) deep outer harbor having a maneuvering anchorage area of 225 acres (910,546 m^2) and protected by two steel sheetpile cellular breakwaters (a main northern arm and a western arm with a single-wall shore connection). Protective structures to the east consisted of a rubble-mound breakwater, a cellular steel sheet-pile breakwater, and a single-wall shore connection.

In the early 1960's, Sverdrup & Parcel and Associates of St. Louis, MO (SPA) developed the initial harbor plan. SPA was assisted in their design by a panel of consultants: Dr. Per Bruun of the Coastal Engineering Laboratory, University of Florida (UF), Mr. Robert Hudson of the U. S. Army Engineer Waterways Experiment Station (WES), and Mr. James Ayers of the Navy Department. At various stages throughout the design period, NCC was involved in review and approval of the breakwater design.

The initial SPA harbor design was tested in 1964 by UF using a three-dimensional (3-D) physical model (1:150 scale). The 3-D model was used to determine optimum breakwater alignment, location and size of the navigation channel, and design and location of harbor elements required to control undesirable wave reflections (University of Florida 1964). The SPA harbor plan was modified using results from the UF model study. In 1965, the modified harbor plan was augmented with a preliminary breakwater cross section. This design was then reviewed by the panel of consultants and further modified during two design conferences.

As a result of Corps of Engineers review, a two-dimensional (2-D) physical model study (1:35 scale) was performed at WES in 1966. The 2-D model was designed to predict breakwater stability and wave transmission through candidate rubble-mound breakwater plans. Results from the 2-D model tests were used to optimize the final breakwater cross section (Jackson 1967). Foundation investigation and design were completed by March 1966.

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[1] A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page xiv.

3-D harbor model study

Approach. In 1964, a 3-D hydraulic model study was conducted by the UF to test the harbor layout from the SPA design (University of Florida 1964). The main objective of the 1:150 scale model was to develop plans that minimized:

a. Navigation risks at the harbor entrance.
b. Wave transmission through the entrance.
c. Reflection of wave energy coming through the entrance by harbor boundaries.
d. Harbor seiching.
e. Adverse effects from wave overtopping and reflection, shoaling, and ice.

The basic harbor plan tested in the model study is essentially the current configuration. The present layout is reduced in area from the original Corps plan to save costs, principally by placing the breakwater in shallower water. Alternative tests focused on design of the harbor entrance. All dimensions in the following discussion refer to prototype, or scaled, dimensions.

Harbor representation. The UF model represented the north breakwater as an impermeable barrier with an external slope of 1V:2H. The jetty head portion of the north breakwater was rounded at the tip. Interior harbor boundaries were vertical, impermeable riparian walls protected at the toe with a rubble-mound slope of 1:2. The north wharf did not contain the projecting quay (Cargill grain dock) built in the actual breakwater configuration. A model ore carrier was built based on dimensions (27,540 tons and 626 ft (25,000 mt and 191 m) in length) from a vessel owned by Bethlehem Steel Company. The model vessel was placed at various locations to monitor moored ship motion by storm surges. Details on its mooring arrangement were not provided. Currents and ship response were observed visually.

During the study, the geometry of the north breakwater was modified in an attempt to reduce the resonant amplification of the harbor basin. The west end of the north breakwater was rotated, hence altering its orientation to the west slip.

Monochromatic wind waves and long waves were generated with a movable flutter-type wave maker. Limited tests were done using multiple-period wave trains between 5.4 and 9.8 sec, but the results "... showed no evidence of harmful effects..." so regular uniform waves were used for testing. Little information was provided on how the waves were measured.

Wind waves. Because wave measurements were unavailable, hindcasting techniques were used to establish the design wave height. The selected hindcasting technique was the Bretschneider-revised Sverdrup-Munk method. This technique assumes that wave height and period are functions of the fetch length, wind velocity, and wind duration.

Design wind waves for the 3-D model study were obtained from hindcasting wind data from Duneland Observatory at Ogden Dunes, IN, 2.5 miles (4 km)

west of the harbor, from April 1956 through March 1959 (approximately 3 years), and from June 1961 through May 1963 (approximately 2 years). Because Burns Harbor was located at the south end of Lake Michigan, winds of significance were determined to range in direction from northeast to northwest. Winds from December through March for each year were rejected on the assumption that lake ice would prevent wave formation on the margins of the lake.

Three deep water design waves were designated for the UF study. During the hindcast procedure, some assumptions with regard to wind duration and fetch length were made which led "... to a hindcast wave with somewhat exaggerated dimensions..." Design wave 1 was based on the longest available fetch (300 miles (483 km) from 8 deg 30' true) and the average wind speed exceeding 20 mph (32 km/hr) in the northwest quadrant, or 25 mph (40 km/hr). The resulting hindcast deep-water significant wave was 10 ft (3 m) high with a 9 sec period. Design wave 2 was based on most direct exposure (northeast) and the average of winds exceeding 10 mph (16 km/hr) from this quadrant, resulting in a hindcast deep water significant wave of 3 ft (0.9 m) at 5 sec. Finally, Design wave 3 was the most frequently occurring wave based on a design wind speed of 10 mph (16 km/hr)from the north. The resulting hindcast deepwater significant wave was 2 ft (0.6 m) at 5 sec. All design waves were considered conservative. To obtain the angle of incidence at the harbor, ray diagrams were used to refract the waves from a water depth of 50 to 40 ft (15 to 12 m), the presumed depth of the structure. Presumably, refraction between deep water and 50 ft (15 m) was ignored.

Design waves 2 and 3 were not considered reproducible at the model scale for harbor response tests because surface tension influenced their behavior at their short (< 1 ft (0.3 m)) wavelength. However, Design waves 1 and 2 were used to test the effect of the length and angle of the eastern end of the north breakwater. The incident waves and periods modeled for harbor response were: 8.5 ft (2.6 m)at 5.4 sec; 7.5 ft (2.3 m) at 7.4 sec; and 6.2 ft (1.9 m) at 9.8 sec. The report (University of Florida 1964) is not clear on why these selections were different from the design wave, except for the earlier mention of surface tension effects.

Long waves. In this study, particular interest was given to long waves below 3 min because the resonant periods of the vessels assumed to frequent the harbor are generally less than 2 min. This range covered all fundamental and higher harmonics of the basin. Long waves caused from storm surge (having periods up to 3 min) were considered rare in Lake Michigan, though a record-setting storm in 1963 causing oscillations on the order of 0.5 ft (0.2 m) was cited. Surge periods tested ranged from 50 sec to 2.5 min. Incident amplitudes ranged from 1.5 to 3 ft (0.5 to 0.9 m), which were considered exaggerated by about a factor of 10 over expected values. Incident amplitudes were exaggerated to permit measurement with existing, parallel-wire model wave gages. The amplitude exaggeration affected surge velocity in the model which, in turn, affected the period of basin resonance. However, "... no significant effect was expected by the order of distortion of the surge amplitude..." Lake seiches with periods on the

order of 15 to 70 min were assumed to produce currents through the harbor entrance, but not induce significant ship response.

Littoral transport. Net littoral sediment drift was assumed to be toward the west on the order of 27,000 cu yd (20,000 cu m)/year (based on adjacent accretion patterns, dredging patterns of nearby harbors, etc.). This was not considered a potential problem with respect to shoaling in the channel because the planned landfill to the east of the entrance served as an effective littoral barrier, and the planned entrance depth of 34 ft (10 m) was 4 ft (1.2 m) deeper than required.

Lake water levels. A water level study was conducted based on records dating back to 1860. In Lake Michigan, water levels varied seasonally and were higher (averaging between 1 and 2 ft (0.3 and 0.6 m) during summer months than in winter months. This seasonal fluctuation was mainly due to precipitation. Cyclic fluctuations were also observed, but theories as to their occurrence were inconclusive. The water level used in the model was 3.1 ft (0.9 m) low water datum (LWD), based on the long-term average for Lake Michigan of 2.1 ft (0.6 m) plus an assumed surge of 1 ft (0.3 m) due to a storm of "... moderate intensity in this area" with a frequency of occurrence of once a month.

Results and recommendations. Tests were conducted to optimize the length, orientation, head geometry, height, and slope of the north breakwater with regard to response to wind waves at the entrance and interior wharfs. The recommended length of the north breakwater tip (that portion projecting eastward of the eastern boundary of the harbor basin) was 700 ft (213 m). One of the principal results of the model study was the recommended harbor entrance design: a 10-deg rotation of the eastern tip of the breakwater, and a curved, revetted wave absorber landward of the entrance channel, offset landward by 645 ft (197 m) from the outer limit wall to the east. In addition, a "streamlined," rounded head configuration with a shallow (about 1:3) lakeward slope was suggested.

The slope on the remainder of the north breakwater was considered satisfactory at 1:2, principally in regard to reflection-caused problems involving small craft navigating near the structure. Of note is the suggestion to place rock on the outer slope so as to "... display maximum degree of permeability and stability simultaneously..." with the longest axis of the rock perpendicular to the breakwater. A crest elevation of +12 ft (+3.7 m) was considered adequate to prevent overtopping. However, the smooth impermeable walls were not expected to accurately simulate wave reflection from or transmission through and over the north breakwater.

Wind wave heights in the harbor typically were 15 to 25 percent of the incident height, with the exception of locally higher values near corners. Nine-sec waves (which were considered infrequent) tended to produce a standing oscillation with an amplitude on the order of 10 percent of the incident amplitude. Wind waves did not measurably displace the model vessel.

The model detected practically no resonance in the east-west direction. Seiche in the north-south direction in the east and west slips was observed at the second (110 - 120 sec), third (70 -80 sec), and fourth (50 -60 sec) harmonics. Maximum resonance was observed in the second harmonics at an amplification factor of 160 to 200 percent. Amplification factors for the third and fourth harmonics were below 150 percent. Vessel motion at the north wharf was negligible. In the east and west slips, north-south vessel excursions ranged from 3 to 9 ft (0.9 to 2.7 m) for an incident amplitude of 0.3 ft (0.1 m). This displacement by storm surge was considered to be of little consequence, although it was recommended that the mooring system of a vessel be adjusted as a "precaution against a storm surge of exceptional intensity" (> 0.3 ft (0.1 m)). Modifying the harbor geometry by straightening the bend at the west end of the north breakwater did not affect harbor oscillations. From these observations, it was concluded that seiche oscillation posed no danger to the harbor.

Deposition in the entrance was considered "... of no immediate concern" due to the existing depths. Erosional effects on the downdrift shoreline were anticipated at about 27,000 cu yd (20,000 cu m)/yr. Bypassing, or construction of protective structures for the beach to the west were suggested. Ice was anticipated to jam the entrance, but this was considered to be infrequent during the navigation season.

The Lake Carriers Association accepted the reduced area and entrance design as suitable from a navigation standpoint. The alignment of the breakwater, particularly the 10-deg rotation in the easterly section, generated questions and discussion since it would complicate horizontal control during construction, but it was eventually accepted by the Corps.

Design and performance criteria

Following the UF model study, design conditions and breakwater parameters were defined. The following information was taken from Indiana Port Commission 1966.

Wind data. A second wave analysis was calculated using wind records from the Duneland Observatory. The total period of record for the data was 9 years and 7 months from April 1956 through July 1965.

Fetch distance. Effective fetch for wave generation at the project site was determined from a technique described in a technical memorandum by the Beach Erosion Board (1954). This technique reduces the actual fetch distance to an effective fetch based on the width of the water body. The largest calculated fetch, 150 miles (241 km), was used in the wave generation analysis. The original fetch distances are approximately half of what they would be if calculated using present day methods. The current recommended fetch delineation procedure involves constructing nine radials from the point of interest at 3-deg intervals and averaging those distances for the final applicable fetch (*Shore Protection Manual* 1984).

Ice cover. Generated wave heights were only representative of a portion of a typical year. It was assumed that from December through March the ice cover in the lake would prevent waves from impacting the breakwater. The following statement explained the assumption:

> It is felt that during much of the winter season, portions of the lake are covered with ice and fetch areas are limited considerably. In addition, the coast area of the lake is covered with ice, and, even though waves are generated in offshore area, they never reach the shore, being interrupted by the ice around the rim of the lake. Omission of waves occurring in the winter months seems reasonable.

Crown elevation. Non-breaking wave conditions were assumed for the design because of the water depth at the project site (approximately 45 ft (14 m)). The required crown height was identified as that crest elevation which produced the situation "... if overtopping occurs, generated waves in the protected harbor will not exceed allowable limits." Attention was also given to possible breakdown of the structure due to overtopping. A crown elevation 1.2 times the design wave height above the still-water level was recommended to prevent excessive damage to the backside.

Crown width. Recommended minimum crown width was three stone widths. Design of the breakwater sideslopes recognized that while flatter slopes are more stable, they may not be economically practicable in deep water. The direct relationship between slope and required stone size was also noted.

Armor unit material. General design of the rubble-mound structure explored several armor unit types including rock, tribars, and tetrapods. The intent of the breakwater design was to make maximum use of available stone at least cost and at the same time provide an adequate structure that would meet acceptable criteria. Bedford limestone was chosen as the most economical stone source. This stone is characterized by a low specific weight (145 pcf (2323 kg/m^3)) and a regular, rectangular shape.

Layer design. Design of the primary cover layer used the Hudson Equation as developed by WES:

$$W = \frac{\Gamma_r H^3}{K_D (S - 1) \cot \alpha} \tag{1-1}$$

where:

W = weight of the armor unit, lb
Γ_r = unit weight of armor unit, pcf
H = design wave height, ft
K_D = stability coefficient
S = specific gravity of armor unit
α = slope of breakwater surface with horizontal

A stability coefficient of $K_D = 3.5$ was taken from Engineer Manual 1110-2-2904 (see Headquarters, Department of the Army 1986 in references) and used for a two-layer design of random stone placement. The areal extent of the primary cover layer started at *-H* below the still-water level on the lakeside, extending over the crest and down to the still-water level on the harborside.

For the secondary cover layer, the weight of armor units between *-H* and -1.5*H* on the lakeward side was recommended as no less than 0.5*W*. These stones were extended to -2*H* on the lakeside for conservative design. Below -2*H* the weight of the armor unit was reduced to *W*/10. On the harborside, armor units of weight 0.5*W* were recommended from 0 to *-H*. Below *-H* the weight *W*/10 was recommended.

Two layers of *W*/10 stone were recommended for the underlayer design, ranging from the bottom of the crest armor units to -1.5*H*. The second underlayer consisted of two *W*/200 stone layers. The original core design recommended the use of sand in an attempt to utilize materials already available at the site.

Damage criteria. To determine the proper relationship between first cost and future maintenance cost, an estimate of the expected damages was made using damage criteria as established by WES. The results give a range of percentages of damage for various ratios of experienced wave heights to design wave height.

Breakwater failure. Four different modes of breakwater failure were identified as follows:

a. Sliding.
b. Lifting - individual armor units are lifted and displaced from position by wave action.
c. Impact - capstones are lifted and pushed or rolled over the breakwater crown.
d. Failure of the foundation soil.

Design conferences

Two design conferences were held in October 1965 to discuss the rubblemound breakwater design. Appendix 1A contains memorandums documenting both design conferences. The memorandums in Appendix 1A are primarily composed of comments by the consultants and a final consensus on design guidelines. Topics discussed during the conferences included selection of the design wave height and water level, wind records, crown elevation, cross section layers and materials, slope and gradation of the breakwater, and economics. This section summarizes the design conference discussions.

Summary. The first design conference was held on 3 October 1965 with the following parties represented:

a. Indiana Port Commission
b. Klein & Kuhn, Industrial Realtors
c. SPA
d. Consultants:
   1. WES (Mr. Robert Hudson)
   2. UF (Dr. Per Bruun)
   3. Navy Department (Mr. James Ayers)

Following the conclusion of the first conference, SPA modified their rubblemound breakwater design and issued the revised design documentation to the consultants for review. A second design conference was held on 29 October 1965 with participants from the first design conference, and representatives from NCC, Bethlehem Steel Corporation, and Midwest Steel Corporation.

The majority of comments during both conferences, as indicated by the minutes, were provided by Messrs. Hudson and Ayers, and Dr. Bruun.

Wind records. Comments were provided on wind records used to determine the design wave height. Mr. Hudson considered the approximately 10-year record of wind data used to determine critical design winds at the site too short a period of record for determination of a design wave.

Dr. Bruun stated that the Ogden Dunes wind data "indicated considerable discrepancy" when compared to Chicago Weather Bureau records. The cause of the discrepancy was "due to a combined effect of a change in type of wind instruments and level of instruments" and it was stated that corrections could be applied which would reduce the wind speeds used in the design wave analysis, thereby reducing the design wave height.

With regard to selection of the design wind conditions in which only the ice-free portion of the year was considered, Mr. Hudson stated "it would be preferable to assume that severe winds occurring during the ice season could occur also during the navigation season."

Concern was expressed over the calculated fetch length of 150 miles (241 km) used in the original design wave height analysis. Mr. Hudson stated that the longest (straight-line) measured length of water in the lake was about 300 miles (483 km)and recommended a design fetch length of 225 miles (362 km).

Design wave height. Much of the discussion centered around the design wave height. The opinions of the consultants varied along with the different methods of calculating the design wave height. Mr. Hudson recommended a conservative design wave height of 16.5 ft (5 m), based on a modified Beach Erosion Board curve incorporating shoaling and refraction, that predicted a return interval of 25 years for a deepwater wave of 18 ft (5.5 m) in Lake Michigan at Chicago. Dr. Bruun recommended a design wave height of 10.5 ft (3.2 m), which was calculated using the Bretschneider method and using corrected wind records. Dr. Bruun felt that the 10.5-ft (3.2-m) height would "permit no damage and no

overtopping." Mr. Ayers recommended a height of 12 ft (3.7 m) based on statistical information from the Beach Erosion Board (1953). Economics of damage versus initial cost were also discussed with regard to the design wave.

During the second conference, Mr. Hudson used the design fetch length of 225 miles (362 km) in the Bretchneider curves and came up with an approximately 13-ft (4 m) significant wave height for stability computations. Mr. Hudson also suggested that crown height could be based on an 11-ft (3.4 m) design wave. Mr. Ayers specifically expressed agreement with Mr. Hudson's recommendations.

Still-water level. Mr. Hudson and Dr. Bruun agreed that the still-water level upon which the design wave was to be superimposed should be 3.0 ft (0.9 m) LWD (average lake level + wind setup of 1 ft based on a 1-month frequency). Statements were made to point out that the combined effect of velocity and hydrostatic pressure is greatest at still-water level.

Breakwater crown. Dr. Bruun recommended a crown elevation of 13 ft (4 m), based on overtopping (13 ft (4 m) on the harborside and 12 ft (3.7 m) on lakeside). Mr. Hudson stated that the establishment of an optimum crest elevation should take into consideration an allowance for some damage. He recommended a crown elevation of $1.0H$ above the still-water level rather than $1.2H$ as previously stated. Other suggestions were made that included the placement of wave screens on the crown to reduce overtopping and putting a concrete cap on the crest to "glue" the top together.

One layer or two? Throughout the conference discussions, the participants expressed concerns about the use of one layer in the design due to inadequate interlocking of regularly-cut armor stone and the potential vulnerability of one layer on an overtopped crest. The two-layer design, while inherently more conservative, raised questions with respect to the viability of randomly placing two layers of regularly-cut stone. However, at the conclusion of the conference, the participants decided on a two-layer armor stone construction extending down to $-H$ below the water level on the lakeside.

Sand core. Protection of the proposed sand core was discussed. Filter layers were discussed by conference participants, and Dr. Bruun suggested using polyvinyl sheets to confine the sand and prevent it from entering the rock layers. Mr. Hudson stated "it would be preferable not to use sand." Locally available blast-furnace rock (fluxstone) was suggested as a more economic alternative for the core layer. In the second conference, Dr. Bruun concurred with Mr. Hudson's preference by stating "in view of the difficulty of placing good filter layers needed to protect the sand core and since an economical rock core material is available, it is recommended the sand core section be eliminated from consideration."

Slope. Slope of the breakwater was reviewed and opinions varied between the participants. Mr. Hudson and Dr. Bruun recommended a slope of 1:2 on the breakwater to reduce wave reflection because of the "...considerable number of

pleasure craft operating in the area..." near the breakwater. Mr. Ayers argued for the sake of economics:

> To require a flatter, 1:2 slope requires additional effort on the part of the contractor in placement. There is heavy enough rock available so that it could be placed on a 1:1.5 slope. Effort should be made to utilize the full output of the quarry for maximum economy and ease of placement. If a wider gradation in the armor can be used, higher costs will be avoided.

Slope geometry was also considered by Dr. Bruun who stated "the most suitable geometry does not use the same slope all the way down. An "S-shaped slope line is recommended..."

Armor material. The attributes of Bedford limestone were discussed. Disadvantages of the limestone included the rectangular or cubical shapes and the low specific gravity of the stone. Also, pell-mell arrangement of Bedford limestone in one layer was not recommended because of instability. Mr. Hudson pointed out that pell-mell arrangement of Bedford limestone in two layers was difficult because the rectangular shapes are not "keyed" in. Recommendations were made as to the usage of tribars and tetrapods and their possible locations and arrangements (either in one or two layers). Mr. Ayers suggested "preparation of two designs, one with stone armor and the other with manufactured units, allowing the Contractor to decide which is most economical..."

Design conference results

The majority of the breakwater design remained similar to the original design with additional definition and decisions in some areas:

a. Ogden Dunes records used for statistical analysis of wind speeds were considered more conservative than the Chicago Weather Bureau records and representative of the long-range record in the area.

b. Wave-height frequency curves were revised to incorporate wind speeds for the entire year.

c. Design wave heights were established that provided good structural stability and economy of construction: an 11-ft (3.4 m) design wave for establishment of the crest elevation with respect to design overtopping, and a 13-ft (4 m) design wave for stability computations of armor units.

d. Crest design was based on a no-overtopping condition of the design wave by setting the crest elevation at elevation $+H$ above the design still-water level (+3 ft (0.9 m) LWD). A concrete cap was discussed as a method to both increase the crest elevation and provide a roadway in the case of frequent maintenance. Since frequent maintenance was not anticipated, a concrete cap was not recommended.

e. Armor stone placement by orienting the longest axis of the stone perpendicular to the breakwater surface was not considered warranted on the Burns Harbor breakwater because a large percent of the armor stones must have one long axis, the selected stone bordered on being smooth, and the placement technique would have to be rigidly controlled, above and below the waterline. Selection of rock size was based on the Hudson equation and availability of material in the vicinity of the project.

f. Some cross-sectional dimensions were further defined. Secondary layer armor units would be extended to an elevation of 10 ft (3 m) below that of the primary layer on the lakeside and to an elevation of 10 ft (3 m) below 0.0 LWD on the harborside. From the bottom of the secondary layer to about 5 ft (1.5 m) from the lake bottom, the underlayer would be composed of two layers of $W/10$ stone. The bottom underlayer was specified as a 3-ft (0.9-m) layer of $W/200$ stone.

g. Damage criteria for the structure were re-examined to incorporate information from damage tests at WES (Hudson 1961). The WES tests were conducted on a breakwater composed entirely of armor material above a point $-H$ below still water level. Reported damage percentages were based upon the entire volume of armor material. For the proposed Burns Harbor breakwater (composed of small core material protected by two layers of heavy armor) it was necessary to revise the WES damage percentages so that they correctly reflected the anticipated damage on the relatively smaller total volume of armor material. Further calculations to determine percent damages for wave heights above the design wave heights were made as well.

h. A rock core (blast furnace stone or similar material) was to be used instead of sand throughout the full length of the north breakwater and west outer bulkhead.

i. Design plans should include three alternative types of armor (tribars, tetrapods, and natural stone) with the choice being left to the contractor. The entire north breakwater and west outer bulkhead were to be bid as one lump sum with the contractor divulging his choice of armor after contract award.

WES 2-D breakwater model study

During the design conferences, questions regarding both the stability and transmission characteristics of the proposed cross section were generated because of the multi-layered design and random placement of armor units. In 1966, NCC requested that WES conduct a 2-D physical model study to verify the stability of the proposed cross section and investigate wave energy transmission through and over the structure. Armor unit stability was explored by random and uniform placement of armor units on the model. In addition, the study examined optimal use of the different sizes of limestone available in the Burns Harbor vicinity. This

section summarizes the 2-D model study and its results. A complete description of the model study is provided in Jackson (1967).

Approach. The WES study was performed using a 1:35 scale physical model of the Burns Harbor breakwater cross section design. The model was constructed in a concrete flume 119 ft (36 m) long, 5 ft (1.5 m) wide, and 4 ft (1.2 m) deep. Model scale was selected based on the size of available model armor units to represent the prototype armor units and the water depth at the toe of the structure. Model armor units consisted of limestone blocks (145 pcf (2323 kg/m^3)) and tribar units. For most tests, the armor units were placed by hand (one at a time) either randomly or uniformly. No attempt was made to interlock armor units. During one of the tests, armor units were dumped from a shovel.

Breakwater stability and wave transmission tests were conducted by subjecting a given breakwater to attack by waves of specified height (ranging from 5 to 20 ft (1.5 to 6.1 m)) and period (7, 9, or 11 sec) for prototype time intervals up to 5.7 hr. A 13-ft (4 m) wave height was selected to represent the prototype design wave. Waves were generated by a plunger-type wave machine located at the opposite end of the flume. The success of the cross section (behavior during wave attack and extent of damage) was determined visually, as was the no-damage design wave (i.e. the largest waves that did not remove armor units from the test section). Two still-water levels, 0 and 4 ft (1.2 m) LWD, provided water depths at the toe of the structure of 43 and 47 ft (13.1 and 14.3 m), respectively. The still water level of 4 ft (1.2 m) LWD was used for the majority of tests.

Wave heights were measured by electrical wave gages in front of the breakwater and at two locations behind the breakwater. Behind the breakwater, wave heights were measured at distances from the center line of the breakwater of one wavelength (*L*) and one-half wavelength (*L/2*) shoreward, where wavelength was calculated from the wave period for the depth of water at the toe of the breakwater.

Results. Eight breakwater plans were modeled during the study. Plan 1 addressed an alternative two-layer design using tribar armor units. Plan 2 was the SPA preliminary design basically consisting of: two layers of 10- to 16-ton (9,000 to 15,000 mt) (W stone), random placed limestone blocks on the lakeside slope and crown extending down to -13 ft (4 m) LWD, a crown elevation of +14 ft (4.3 m) LWD, and a sideslope on both the lakeside and harborside of 1:1.5. Smaller stone (W/2 stone)) were placed from -13 to -27 ft (-4 m to -8.2 m) LWD (Figure 1-1). Plans 3 through 8 were different configurations of Plan 2. The Plan 8 cross section is illustrated in Figure 1-2.

Plan 1. It was determined from tests that the tribar armor layer in Plan 1 would be stable for the selected design wave height of 13 ft (4 m), although stability could be improved by modifications on both the lakeside and harborside. No additional tests were conducted using the tribar armor layer.

Plan 2. Tests showed that Plan 2 was stable for 13-ft (4-m)waves but damaged by 15-ft (4.6-m) waves. For the 13-ft (4-m) design wave the maximum

transmitted wave height was found to be about 2 ft (0.6 m). The data also show that the heights of the transmitted waves were reduced about 50 percent when the still-water level was lowered from +4 (+1.2 m) to 0 ft LWD.

Plan 3. The configuration of W/2 stones on the lakeside slope was changed from a triangular shape (tested in Plan 2) to a rectangular shape. The modified cross section improved the stability of the W/2 stones at both the +4- (+1.2-m and 0-ft water levels, and Plan 3 was considered stable for 15-ft (4.6-m) waves.

Plans 4, 5, 6. Tests of Plans 4, 5, and 6 investigated the effect of uniform placement of the armor stones both as a design element and as a possible random occurrence. All uniform placement was on the lakeside. Plan 4 armor stones above 0 ft LWD were placed uniformly. Uniform placement of the armor stone above -13 ft (-4 m) LWD was studied in Plan 5, and in Plan 6, armor stone was uniformly placed in scattered areas of the breakwater. Stability results from Plans 4 and 6 were similar to those from Plan 2; uniform placement neither increased nor decreased stability. Plan 5 resulted in "measurably improved stability" of the W stone, when compared to Plan 2 results. However, during Plan 5 tests it was observed that overtopping and hence wave transmission were considerably increased over the smooth surface created by the uniformly-placed armor stones on the lakeside. When wave transmission data for Plan 5 was compared with Plan 2 (random-placed armor units), the maximum transmitted wave height was increased by about 50 percent (from overtopping waves). It was also found in Plan 5 that 25 percent more armor units by count were required to achieve uniform placement.

Plans 7 and 8. These plans were developed following the Plan 2 tests and were designed to "eliminate weak elements of the original (Plan 2) design" such as armor stone instability on the harborside. Other considerations when developing Plans 7 and 8 were to eliminate armor-unit placement problems and develop a stable breakwater with no increase in construction costs. In the Plan 7 design, W stones on the lakeside were extended from -13 ft (-4 m) LWD (in Plan 2) to -27 ft (-8.2 m) LWD. Modifications to the harborside were made by adding a single row of W stone to anchor the toe of the armor stones on the crown. Below the anchor stones, W/2 stone were used to -13 ft (-4 m) LWD. Stability tests on Plan 7 resulted in damage from 13- to 15-ft (4 to 4.6 m) waves on the harborside, although the lakeside remained stable. Wave transmission results were slightly higher than in Plan 2. Plan 7 was considered stable for 11-sec, 15-ft (4.6 m) waves.

The lakeside slope in Plan 8 was the same as in Plan 7. However, the harborside slope was modified by using one layer of W stone from +3 to -13 ft (+0.9 to -4 m) LWD. At high water levels (+4 ft (+1.2 m) LWD) minor damage was sustained on the lakeside, and no damage was observed on the harborside slope. No damage to the breakwater was observed at 0 ft LWD. Plan 8 was considered stable for 11-sec, 15- to 18-ft (4.6 to 5.5 m) waves at both water levels.

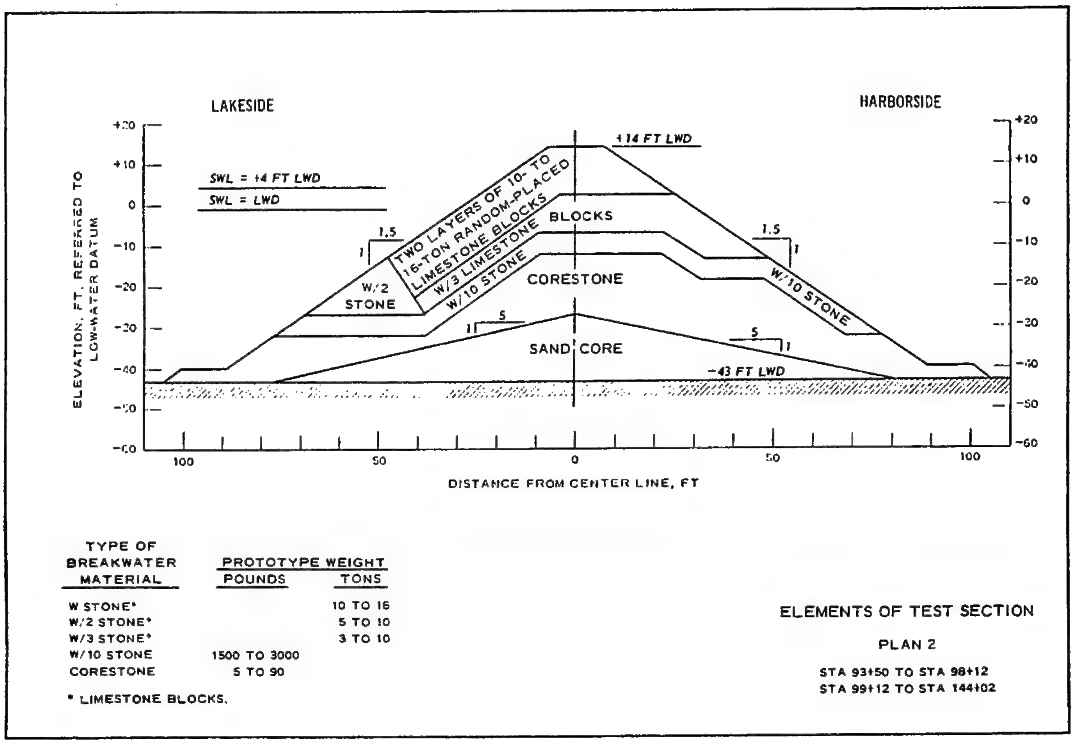


Figure 1-1. Plan 2 model test section

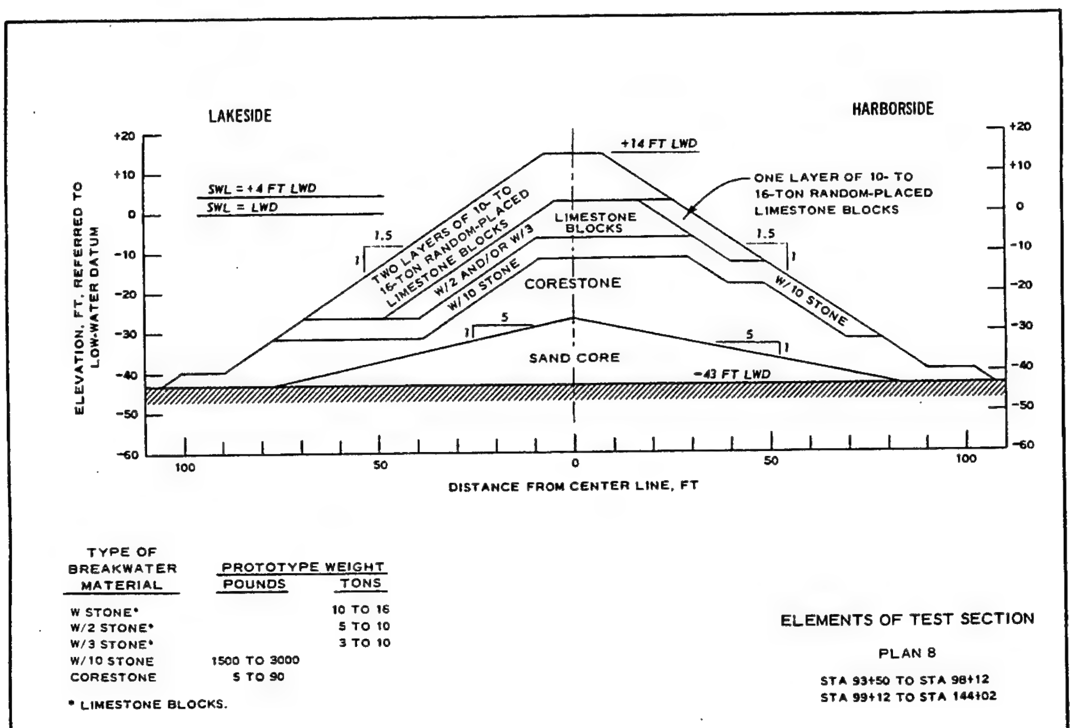


Figure 1-2. Plan 8 model test section

For the 13-ft (4-m) design wave height, the largest Plan 8 transmitted wave was 3.1 ft (0.9 m) at +4 ft (+1.2 m) LWD. Transmission results for waves less than 14 ft (4.3 m) from Plans 2 and 8 for both water levels were similar. The Plan 8 modification to the harborside slope resulted in a considerable increase of transmitted wave heights for 11-sec, 14-ft (4.3-m) or greater wave height. The increase in transmitted wave heights was attributed to increased porosity and smoothness of the armor layer on the harborside.

It was also found that transmitted wave heights for incident waves greater than 12 ft were affected when the water level was lowered from +4 (+1.2 m) to 0 ft LWD; a 25-percent reduction in wave transmission was observed. No appreciable change in transmitted wave heights was observed for incident waves less than 12 ft (3.7 m) at the lower water level.

Thickness and porosity. The experimental shape coefficient, layer thickness, and the porosity of the limestone blocks were also investigated during the study. The armor units used for each test cross section were weighed prior to placement to determine the total weight of the armor layer. Armor layer thickness was measured from soundings taken before and after armor unit placement. These measurements were used to calculate the thickness and percentage of voids in the armor layer. Calculations from Plans 2 through 8 were averaged resulting in a shape coefficient of 1.0, a two-layer armor stone thickness of 11.6 ft (3.5 m), and a porosity of 41 percent for the limestone armor layers.

Study recommendations. As a result of the WES 2-D study, Plan 8 (Figure 1-2) was selected as the optimum breakwater plan for construction because of its improved stability, although transmitted wave energy was greater than in Plan 2. Also, wave transmission data showed that for the selected prototype design wave of 13 ft (4 m), the maximum transmitted wave height would not be greater than about 3 ft (0.9 m). Figure 1-3 is a reproduction of the measured transmitted wave data for Plan 8 from the study.

Foundation

SPA explored the foundation through testing and classification of 14 borings up to 50 ft deep along the planned breakwater alignment. Standard Penetration Test results and Atterberg limits were obtained for all boring sites. Consolidation tests were performed on 12 sub-samples, but information concerning the rate of consolidation was not provided. Design of the foundation was challenging because of the variability in the material properties of the clays, sand, and gravel underlaying the lake bottom, and their distribution throughout the harbor area. The uppermost layer of fine and medium sand ranges from 0 to 8 ft (0 to 2.4 m) thick. Below the sand is a layer of soft, silty clay with some gravel, ranging from 0 to 20 ft (0 to 6.1 m) thick. Lowermost is a glacial till consisting of stiff silty clay, occasionally mixed with sand and gravel, extending to the maximum boring depths.

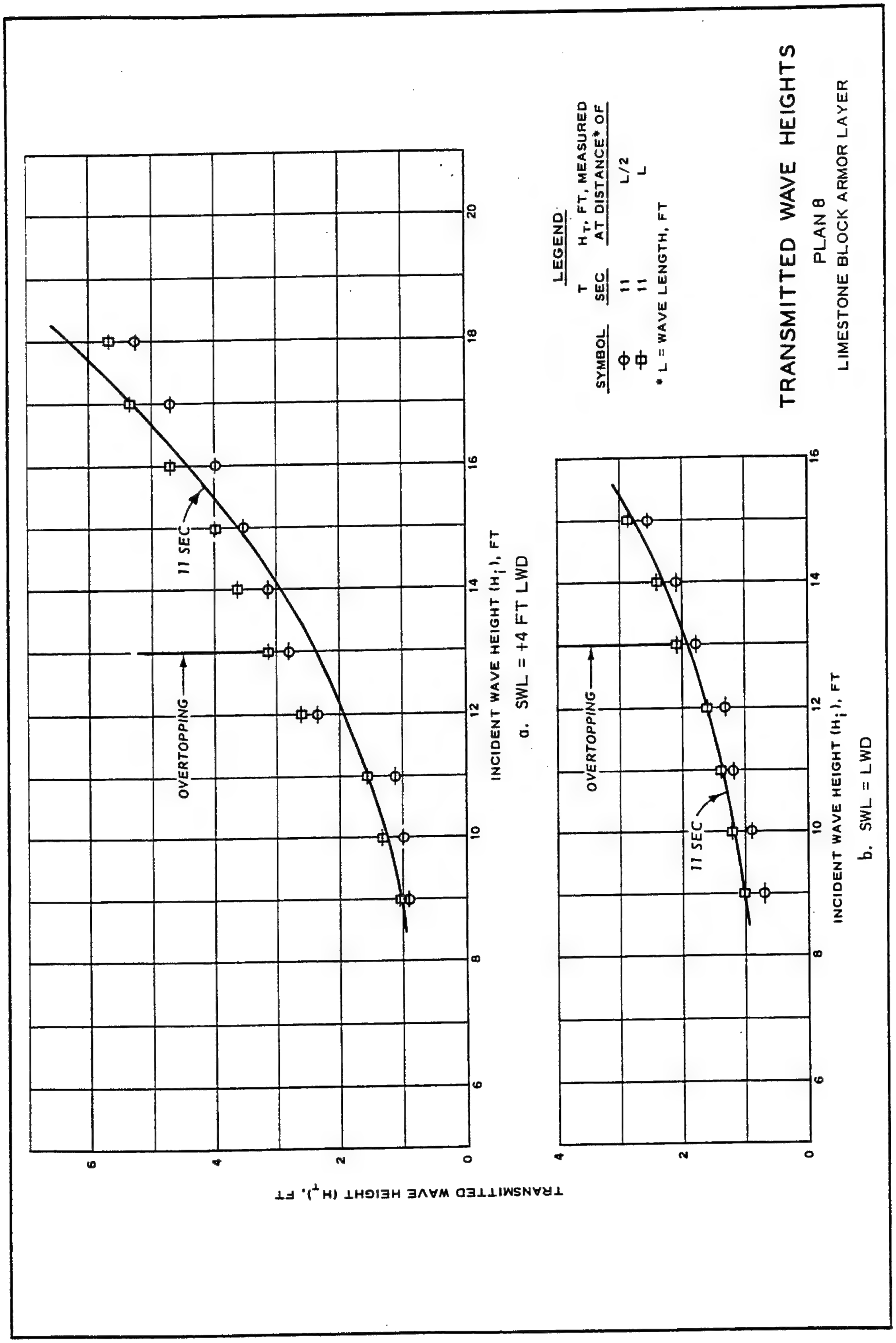


Figure 1-3. Plan 8 transmitted wave heights from 1966 2-D study

No foundation preparation was considered in the initial plan submitted in 1965. The next year, a report prepared by another consultant predicted variable settlement on the order of 2 to 3 ft (0.6 to 0.9 m) would occur from consolidation of the upper 13 ft (4 m) of soil. Consolidation of layers below 13 ft (4 m) was assumed to be negligible. To prevent this predicted settlement, the foundation for the breakwater was prepared by excavating the clay layers to depths varying from 0 to 20 ft (0 to 6.1 m), as determined from analysis of boring logs along the structure's center line, and back-filling the trench with sand prior to core placement. Figure 1-4 illustrates the depth of excavation and the elevation of the sand backfill for each 100-ft section of the breakwater. Station numbers begin at zero at the eastern tip of the breakwater.

Construction History

Breakwater construction commenced on 2 June 1966. The first vessel unloaded cargo in the harbor on 11 September 1969. Construction progressed simultaneously in overlapping stages; excavation of the lakebed to the design depth was the first step, followed by backfilling with sand from the dunes being leveled for construction of port facilities. No information on the placement method for the sand is available. Bedding stone was placed over the sand by conveyor belt. Stone layers were dumped or, for the armor stone, individually placed by crane.

Construction started on the west end of the north breakwater and proceeded eastward; then the western arm was completed. Figure 1-5 represents the sequence of excavation and sand backfill operations based on contractor progress reports. Figures 1-6 and 1-7 are photographs of placement of core stone and armor units, respectively, during construction. Stakes used for position control are visible. Figure 1-8 is a typical cross section from the as-built survey conducted by the construction contractor, and the same section obtained in a 1975 condition survey.

Construction operations were suspended during the periods January through March 1967 and December 1967 through March 1968 because of winter conditions. Excavation and backfill operations were completed in June 1968, and breakwater construction was completed in September 1968. In August 1970, harbor dredging was completed and the official harbor opening was held. Maintenance responsibility for the Federal portion of the harbor (breakwater and channel) was accepted by the Corps on July 1, 1972.

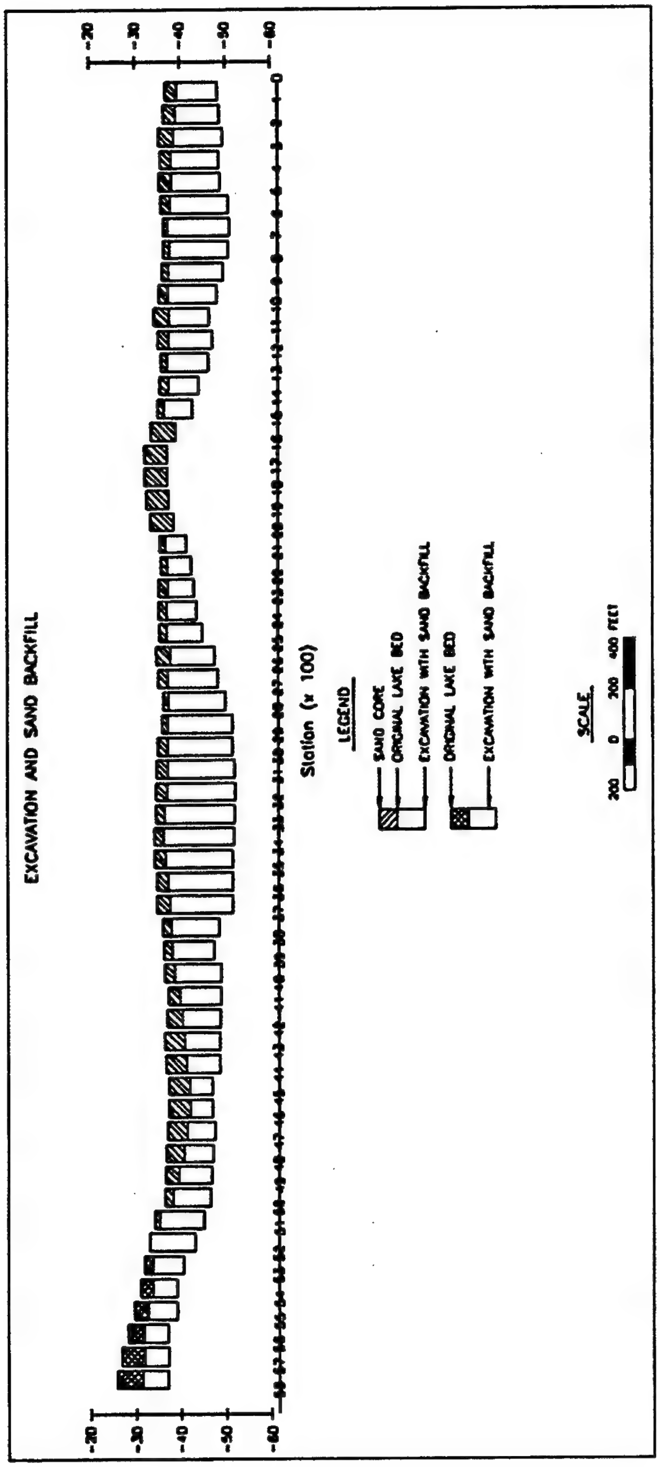
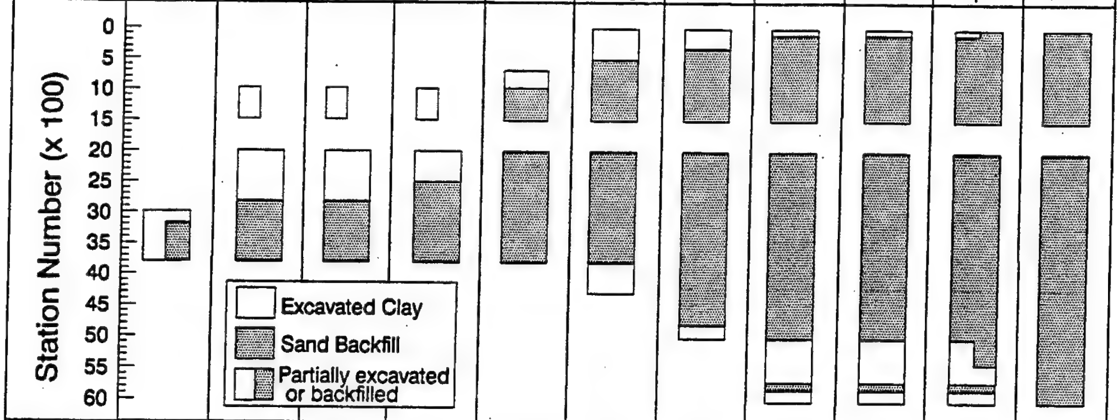


Figure 1-4. Foundation excavation and backfill design

Progress Report #	5, 6	7	8	9	10	11	12	13	14	15	16
Year	1966		1967						1968		
Month	Aug	Dec	Jan-Mar	April	June	Aug	Oct	Nov	Dec-Mar	April	June



Excavated Clay (cu yd)	NA	168,805	No Activity	NA	50,000	81,000	47,000	NA	No Activity	NA	Total NA
Sand Backfill (cu yd)	NA	129,740	No Activity	NA	106,700	58,500	87,800	NA	No Activity	NA	NA

NA = Not Available

Figure 1-5. Excavation and backfill sequence

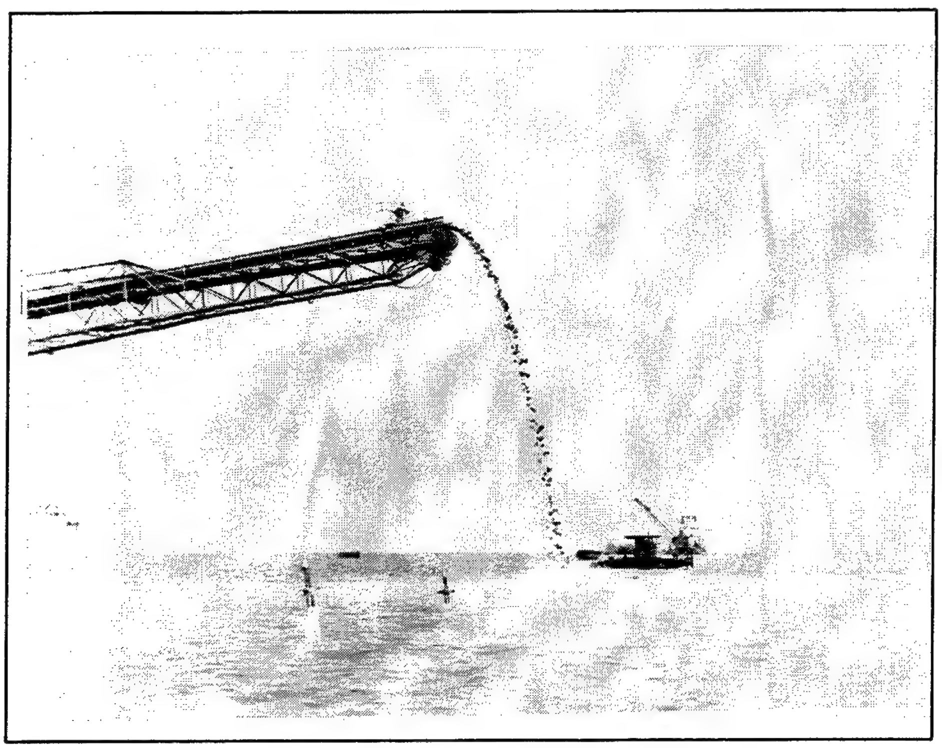


Figure 1-6. Corestone placement by conveyor

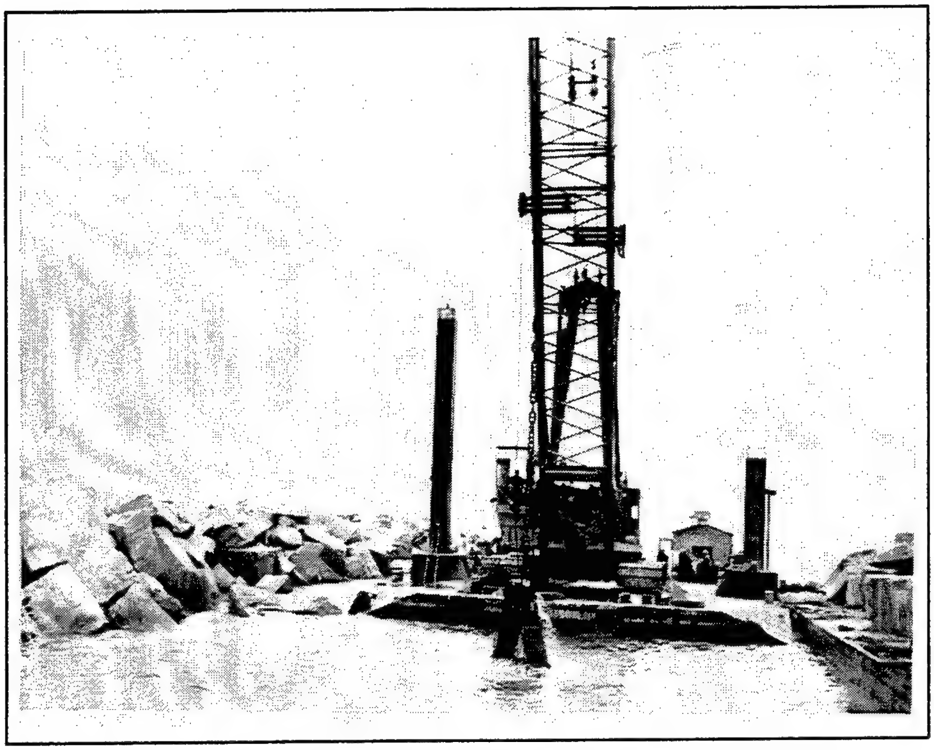


Figure 1-7. Armor stone placement during construction

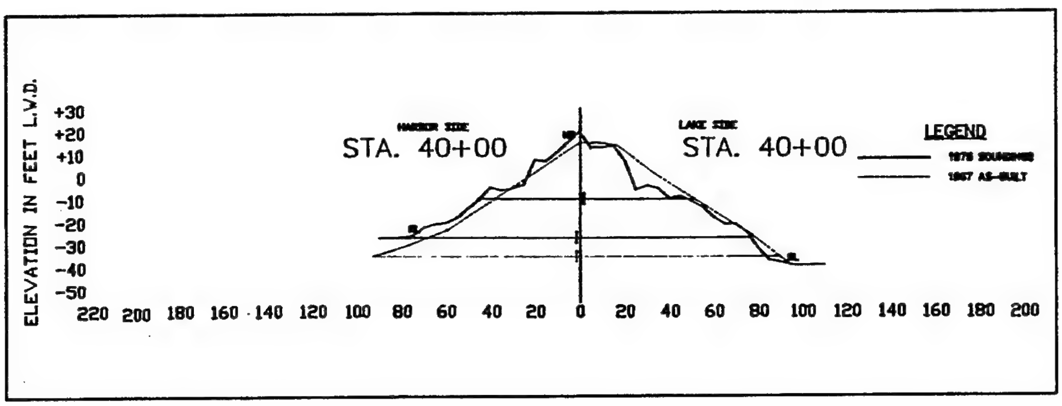


Figure 1-8. Typical as-built cross section

Project Performance

Environmental loading

Wave history. Since construction of the Burns Harbor breakwater, damage from wave loading has occurred during storm events. A storm wave history was developed from a numerical wind and wave hindcast model. The hindcast was conducted by the WES Coastal Engineering Research Center (CERC) Wave Information Study (WIS) and covers 32 years of record (1956 through 1987) for the Great Lakes (Hubertz, Driver, and Reinhard 1991). The nearest WIS station (Station 62) is located approximately 10 n.m. (18.5 km) north of Burns Harbor (Figure 1-9). Data from Station 62 will be described as representative of the incident conditions affecting the project.

Storm events for the 32 years of record were selected from WIS Station 62 data by searching for particular threshold criteria; specifically the occurrence of wind speeds of 20 mph (32.2 km/hr) or greater from the northern quadrant (315 to 45 deg true) for a minimum of 9 hr duration. For the period of record 384 storms met the storm criteria. Wind and wave information were compiled for storm events that met these criteria.

Figure 1-10, which plots the number of storms for each year during the 32-year period, indicates that storms falling in the specified criteria are relatively common with an average occurrence of 12 storms per year. An increase in the number of storms occurred during the period 1973 through 1977 just when it was discovered that the breakwater needed maintenance. The maximum number of storms occurred during 1976. Total storms for each month for all years are plotted in Figure 1-11 which shows that January, February, and March were the most severe storm months for the period 1956 through 1987.

Thirty-two storm events with the greatest deepwater significant wave height were chosen as the maximum storm events. Figure 1-12 illustrates how those maximum storm events were distributed over the 32-year period of record. For years with more than one extreme storm event, only the greatest storm event is shown.

From the period 1967 (during breakwater construction) through 1987, a total of 13 storm events produced waves greater than 13 ft (4 m), exceeding the breakwater's armor stone design criteria. Following completion of the breakwater, the first three storm events exceeding the design wave occurred during the winter/spring seasons of 1973 and 1974. From 1975 through 1987, ten winter storm events occurred that exceeded the design wave height. Waves from

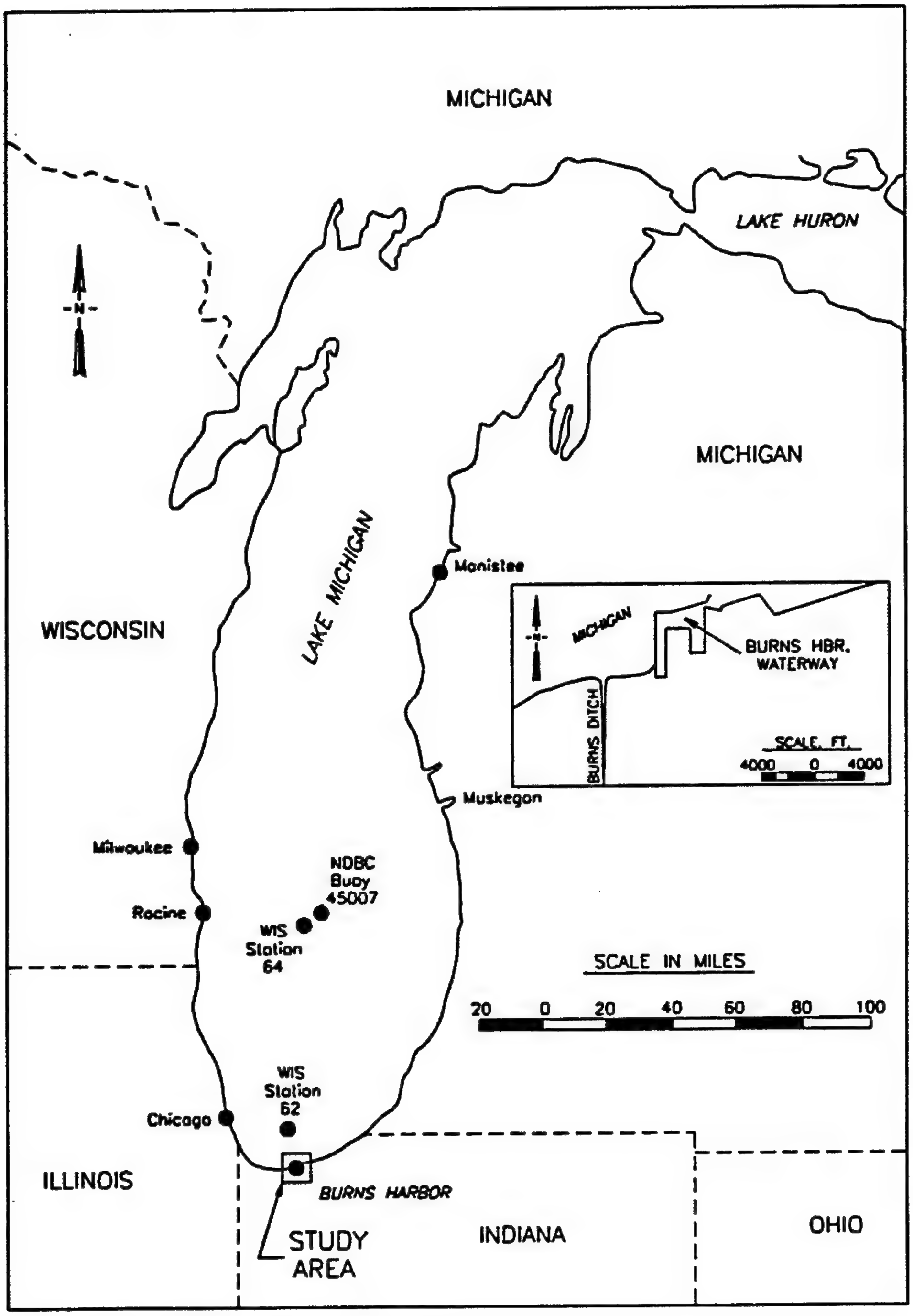


Figure 1-9. Location of WIS Station 62

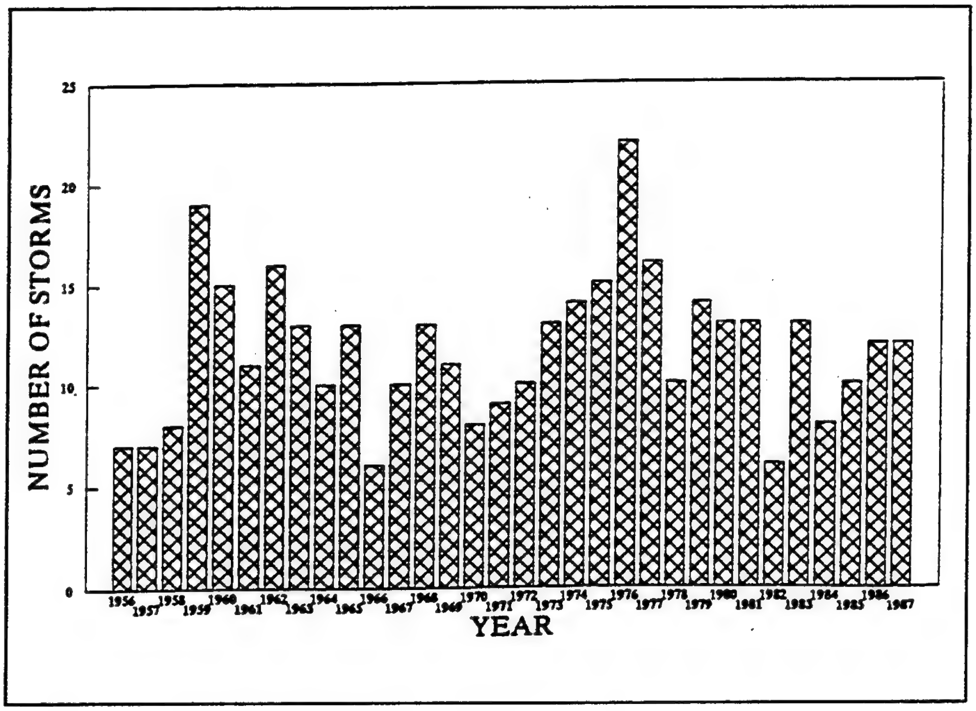


Figure 1-10. Station 62 storm distribution (1956-1987)

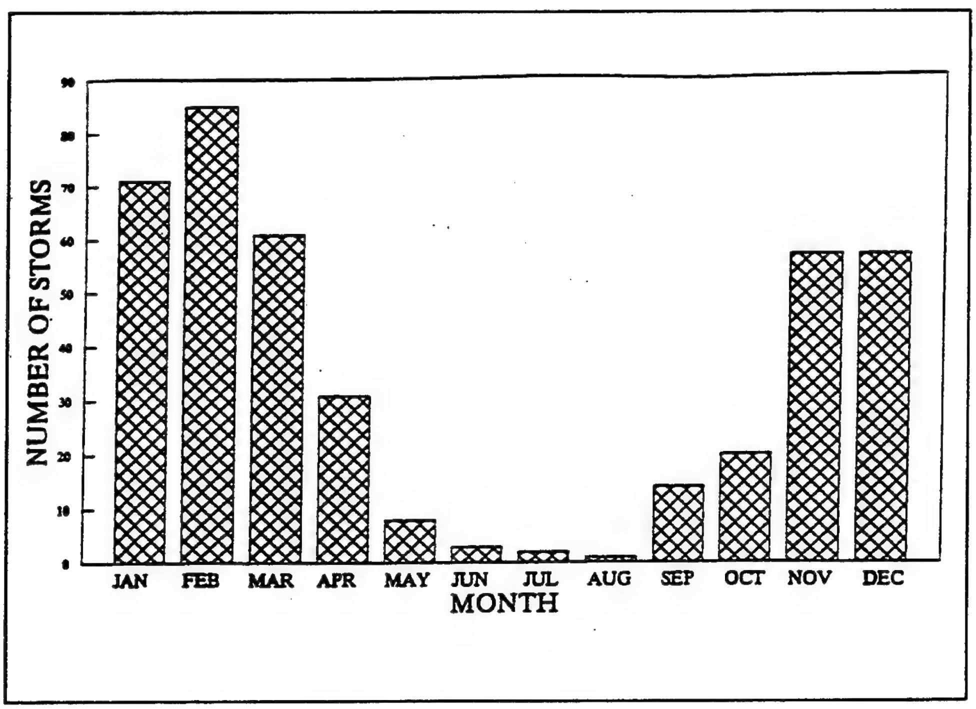


Figure 1-11. Station 62 monthly storm distribution (1956-1987)

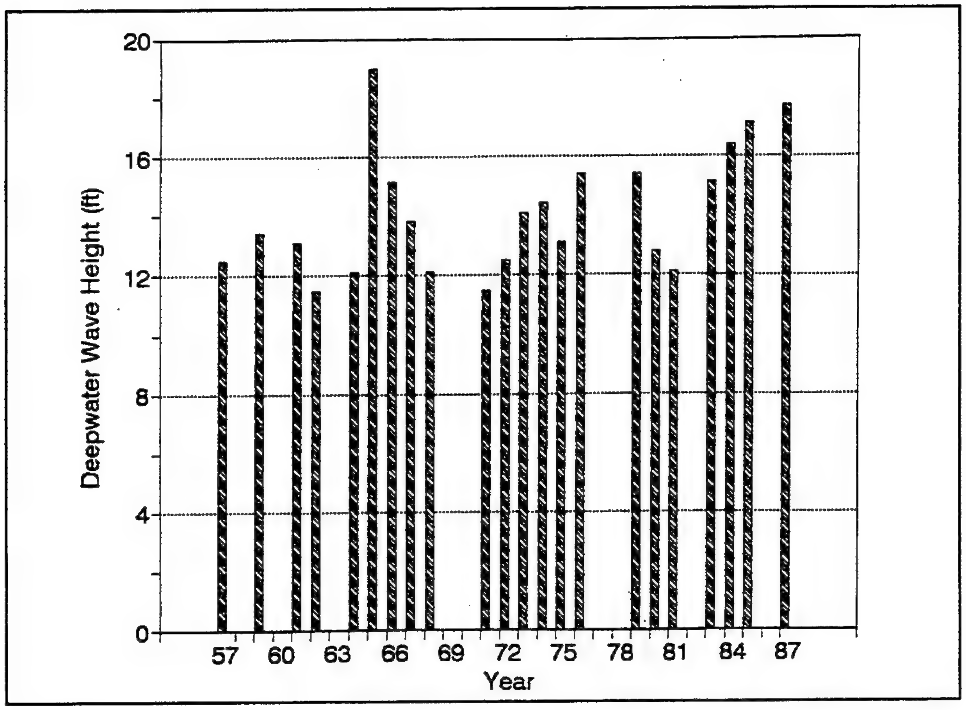


Figure 1-12. Significant storm history (1956-1987)

two of the ten storm events exceeded the design wave height by 20 percent or more. The remaining storm events analyzed for the study period (1967-1987) resulted in wave conditions approaching the 13-ft (4-m) design wave height (> 12 ft). No significant storm events affected the Burns Harbor complex between spring 1987 and winter 1989.

Water levels

Average water levels for Lake Michigan were available from the Calumet Harbor gage No. 7044, operated by the National Oceanic and Atmospheric Administration (NOAA), located approximately 20 miles (32.3 km) northwest of Burns Harbor. The gage is a float-in-stilling-well type that records analog output for postprocessing. Mean water levels, referenced to low water datum (LWD), are calculated over each consecutive 6-min interval, and reports of monthly and annual statistics are published by the U. S. Army Engineer District, Detroit. Figure 1-13 is a time series plot of the annual average, maximum, and minimum lake water levels from the Calumet Harbor gage during the period 1903 to 1992. The data in Figure 1-13 indicate an extreme range of 6.5 ft (2 m), from a low level of -1.45 ft (-0.4 m) LWD in March 1964 to a high of +5.14 ft (+1.6 m) LWD in June 1986. From 1964 to the present, a gradual rise in water levels has occurred, possibly due to the cyclic water level fluctuations previously mentioned (U.S. Army Engineer District, Detroit 1988).

Daily mean and maximum water levels during the 32 storm events described above were extracted from the Calumet Harbor data and are plotted in Figure 1-14. The average hourly and maximum lake levels were +2.78- and

+4.09-ft (+0.8- and +1.2-m) LWD, respectively, for 1956 through 1987. However, from 1967 through 1987 (the time period for which Burns Harbor has been in existence), the average mean and maximum lake level were +3.50- and +4.13-ft (+1.1- and +1.3-m) LWD, respectively. The greatest water level (+6.02 ft (+1.8 m) LWD) for that data set occurred in 1987.

Ice cover

Historical records of ice occurrence in Lake Michigan were available from the National Oceanic and Atmospheric Administration National Weather Service for the time period 1973 through 1989. Table 1-1 provides ice information for the Burns Harbor area during the winter season (December-March). Table 1-1 shows that ice conditions during the 16 winter seasons were widely varied, ranging from no ice cover throughout the entire winter to a maximum ice cover of 66.1 percent in the 1977-1978 winter season. It is also noted from Table 1-1 that three consecutive winter seasons during the period 1976 through 1980 had ice coverage exceeding 50 percent. In addition, seven storms occurred during those winter seasons with significant wave heights exceeding the 13-ft (4-m) design wave, although only four occurred while the harbor was somewhat sheltered by ice. Table 1-2 lists the four storm events that occurred during periods of ice cover at Burns Harbor. The average percentage of ice cover days during the winter season of 1973 through 1989 is 34.2 percent.

Damage and maintenance history

The documented history reveals a harbor viewed as a "problem" by port users from an operations perspective, and by NCC from a maintenance perspective, since shortly after construction. A series of letters beginning in 1973 from the Indiana Port Commission repeat complaints of excessive wave action and perceptions of breakwater damage. This suspicion may have been related to an earlier internal memo that cited four examples of failures (Burns Harbor excluded) occurring with two-layer, randomly-placed armor structures in the Great Lakes.

There have been several serious instances of interior damage from wave action: barges have broken their moorings and been damaged, two vessels and two barges have sunk while moored at the Cargill grain dock, and north-facing revetments require frequent repair. Appendix 1B provides a summary of the major damage events within the harbor in letters from the Port Commission and the operator of the grain dock, and a summary of historical breakwater damage obtained from examination of historical photographs. Repairs to the breakwater itself have been much more frequent and costly than anticipated.

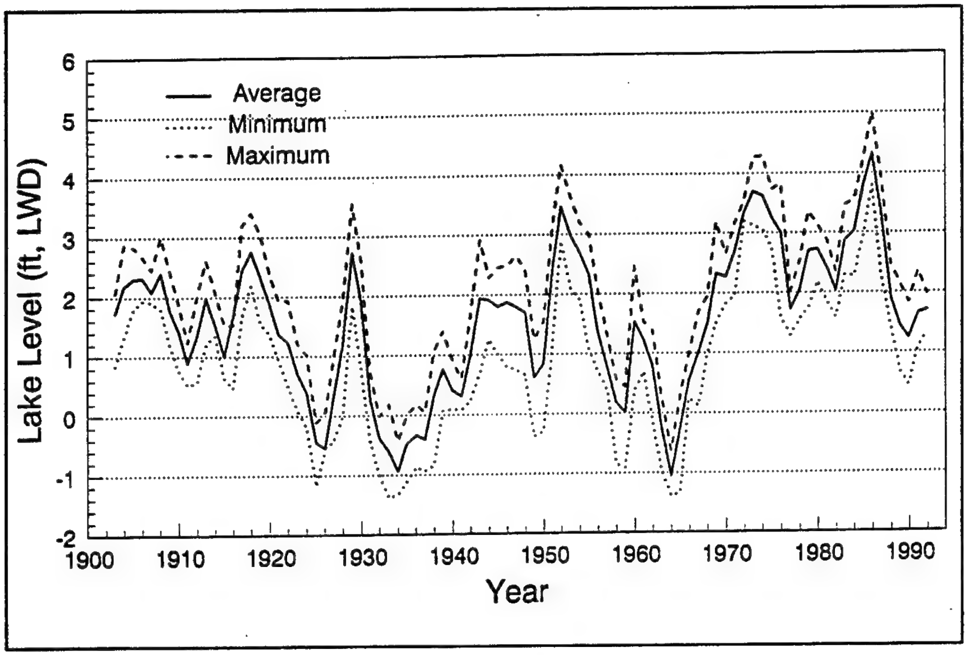


Figure 1-13. Annual water levels

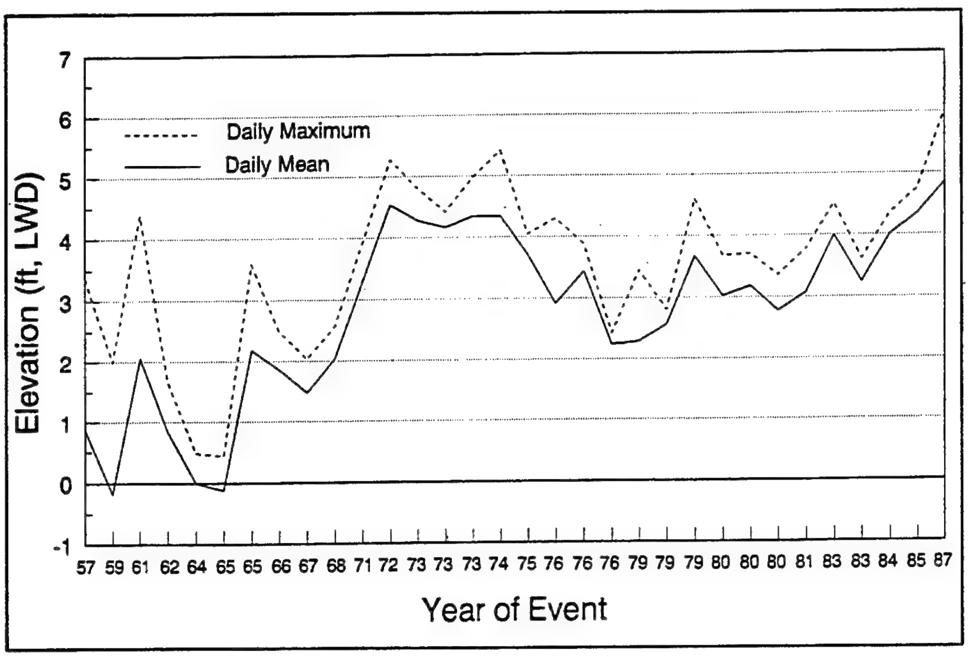


Figure 1-14. Storm event lake levels, 1957-1987

Table 1-1
Ice information for Burns Harbor 1973 - 1989

Winter Seasons (Dec - Mar) years	Ice thickness, in.	Days with ice	
		Total	Percent
1973 - 1974	2 - 6	8	6.6
1974 - 1975	2 - 4	27	22.3
1975 - 1976	2 - 3	34	28.1
1976 - 1977	1 - 8	67	55.4
1977 - 1978	5 - 20	80	66.1
1978 - 1979	3 - 8	71	58.7
1979 - 1980	2 - 8	52	43.0
1980 - 1981	1 - 6	33	27.3
1981 - 1982	2 - 6	46	38.0
1982 - 1983	1 - 2	14	11.6
1983 - 1984	1 - 4	55	45.5
1984 - 1985	1 - 6	69	57.0
1985 - 1986	6 - 11	23	19.0
1986 - 1987	0	0	0.0
1987 - 1988	1 - 6	40	33.1
1988 - 1989	0 - 6	42	34.7

Table 1-2
Ice Conditions for Storm Events

Date of Storm Event	Ice Thickness, in.	Duration of Ice Cover	Quantity of Lake Ice
01 Feb 76	2	07 Jan - 09 Feb 76	Light
14 Jan 79	8	22 Dec 78 - 02 Mar 79	Moderate
26 Feb 79	4	22 Dec 78 - 02 Mar 79	Heavy
12 Feb 85	0-6	23 Jan - 01 Mar 85	Moderate

The earliest damage to the structure is not well documented, but several documents make reference to an event during construction that resulted in damage, necessitating repairs the following year. One of the 32 maximum storm events produced 12.1-ft (3.7-m) (hindcast) waves on 15 December 1968 and could

have been responsible for that damage. In an NCC memorandum dated 12 March 1970, the Burns Harbor breakwater was described as having no damage apparent from shore. The earliest well-documented damage/problems at the harbor are described in a letter dated March 9, 1973, from the Indiana Port Commission (IPC) to the NCC District Engineer. Observations after two severe storms (14.1-ft (4.3-m) waves/29 January; 12.1-ft (3.7-m) waves/15 February 1973) revealed "some" stones lost, and "small gaps" as low as 3 ft (0.9 m) below the design crest. Without additional elucidation, the letter states "... there was indication that there has been some subsidence of the breakwater."

There is no mention of increased wave transmission in this first letter; in fact, the breakwater is described as " ... a very effective barrier to wave action during observed storms." Barely a week later, the U.S. Army Corps of Engineers tug "Moore" was sunk at the south end of the western arm inside the harbor during a severe storm. The hindcast significant wave height on 18 March 1973 was 12.8 ft (3.9 m).

Afterwards, IPC complaints of damage within the harbor and deterioration of the breakwater escalate. Damage is corroborated by a CERC field trip in October 1974, citing "extensive damage" and reduced freeboard. Ice was reported to pile up against the breakwater because of northerly winds and spill over the top. Additional corroboration was given in a memorandum from an NCC field trip in March 1975 which noted "...numerous gaps extending to within 2 to 4 feet (0.6 to 1.2 m) of the lake level." The documentation also indicated that "wave conditions within the harbor were sufficient to break ship mooring lines" and noted that a second large vessel had been sunk while moored in the harbor since 1973. IPC had also reported reoccurring damage to the rubblemound along the north face of the riparian wall. The increasing evidence of structure damage culminated in a request from the NCC Operations Division to the Engineering Division in January 1975 for an investigation.

The requested investigation is described in an unpublished report, "Burns Harbor Indiana - Hydraulic Analysis for Performance of Federal Breakwaters for Period 1967 to 1975." When the first condition surveys, conducted in April 1975, were compared to the as-builts, the problems associated with quantifying structure volume became apparent. Delineating changes, even qualitatively, is extremely problematic for a rubble-mound structure, particularly below the waterline. While there was visible damage to the armor layer, and damaged areas were calculable from the surveys, analysis indicated a net gain in area for the lakeside armor, and a substantial net loss in harborside armor. Survey error was postulated to explain the improbable growth in the lakeside armor (see Chapter 5, this volume for additional discussion of survey problems).

Loss of harborside armor was attributed to survey error, inadequate armor size, damage from overtopping and/or transmission, or settlement. Other pertinent results were that section width at the LWD had reduced an average of 10 ft (3 m), or 18 percent, for the north breakwater, and 6 ft (1.8 m) for the west breakwater, and the average elevation along the north breakwater crest was unchanged from the +14.0-ft (+4.3-m) design elevation. The report concluded:

(a) the structure was exposed to 18-ft (5.5-m) waves since construction, (b) performance of the lakeside verified WES model results showing that the structure is stable for these waves, and (c) the significant damage on the harborside was not predicted.

Armor stone repair on both lakeside and harborside was scheduled for the next 5 years, commencing in the summer of 1975. A report of inspection by the NCC District Geologist in July of 1975 declared the structure to be in better-than-expected condition, but the following summer, the IPC was repeating its request for repair work. Armor stone repair continued until 1978. Annual stone placement during that period on different sections of the breakwater varied from approximately 10,000 to 17,000 tons (9,000 to 15,000 mt) of stone. Most of the maintenance stone was placed on the lakeside below the waterline. No substantial damage was observed on the breakwater during those years of maintenance activities. No stone was placed on the breakwater in 1979. In 1980, a total of 47,000 tons (43,000 mt) of stone was placed both harborside and lakeside of the breakwater (three times the amount of previous maintenance years) due to a 1979 increase in storm severity.

In 1980, the emphasis of maintenance began to shift from transmission reduction to damage reduction of the breakwater itself. This pattern of damage and repair continued through the 1980's with some sections receiving repeated maintenance. Repeated stone placement at previously maintained sections was conducted at three locations along the breakwater: a) northeastern terminus, b) lakeside center portion, and c) lakeside northwest corner. All of the maintenance stone placed on the breakwater was limestone with a specific weight of 145 pcf (2,323 kg/m^3) except for the 1989 repair, in which quartzite with a specific weight of 175 pcf (2,803 kg/m^3) was used. Figure 1-15 records the history of repairs by year, tonnage, and location. The amount of repair stone placed each year and cumulatively is provided in Table 1-3; the total is 78 percent of the original armor amount. Figure 1-16 is a time line providing an overview of wave conditions (from WIS) and major events in the history of the structure.

In addition to continued maintenance activities, damage reports from the IPC to NCC escalated, repeatedly expressing concern over extreme wave conditions during storms in the harbor and citing incidents of rubblemound and bank erosion, damage to the dock area, and excessive ship surge motion, which, in some instances, resulted in the vessel sinking. In 1992, the IPC estimated repair costs of damages from 1970 to 1991 as approximately $1.5 million (not including the costs of the sunken vessels).

In 1984, NCC nominated Burns Harbor for inclusion in the Monitoring Completed Coastal Projects (MCCP) Program. The nomination identified excessive breakwater maintenance and harbor damage as problems, and raised the issue that the design could be inherently deficient. The Burns Harbor nomination was approved in FY85. The original nomination and approval memorandums are provided in Appendix 1C.

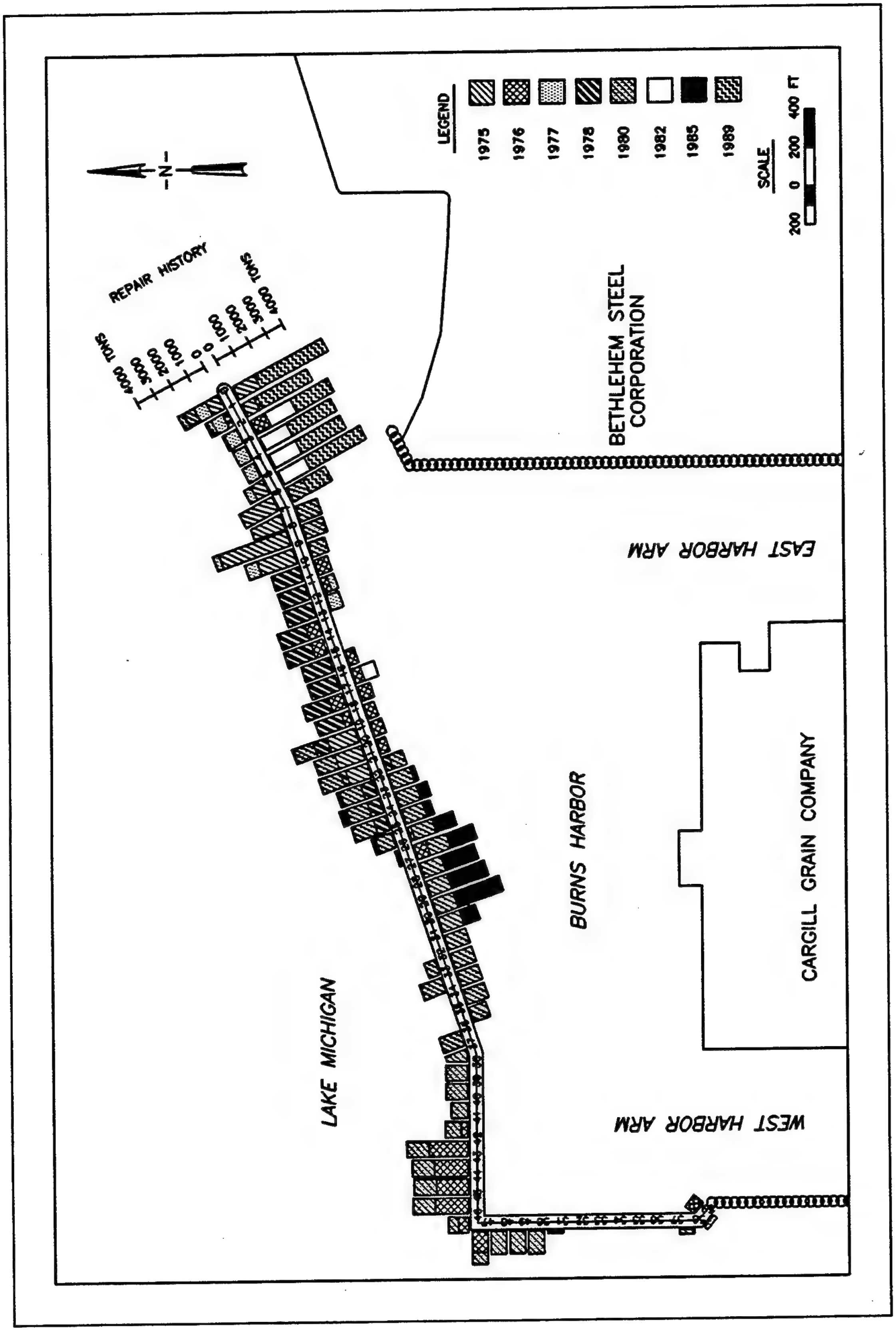


Figure 1-15. Breakwater repair history

Table 1-3
Annual Cumulative Repair Stone Placed, Tons

Year	Harborside	Lakeside	Total
1975	2,028	14,703	16,731
1976	6,463	10,555	17,018
1977	1,373	8,904	10,277
1978	0	14,345	14,345
1980	20,944	26,385	47,329
1982	6,957	0	6,957
1985	11,083	750	11,833
1989	19,477	1,150	20,627
TOTAL	68,325	76,792	145,117

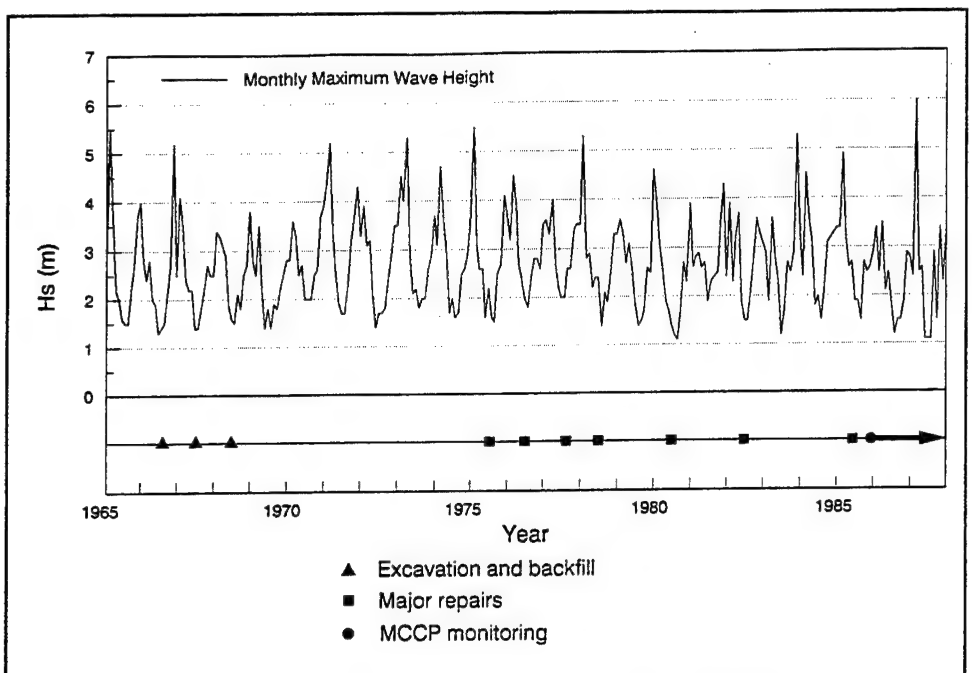


Figure 1-16. Time line of significant events with wave conditions

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Appendix 1A
Design Procedure Documentation

MEMORANDUM FOR RECORD

Revised October 25, 1965
October 12, 1965

PROJECT: Burns Waterway Harbor

SUBJECT: Conference on Rubble Mound Breakwater Design, October 3, 1965

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MEMORANDUM FOR RECORD

Revised October 25, 1965
October 12, 1965

SUMMARY:

1. Mr. McGavock presented a brief history of the Burns Harbor project, beginning with the creation of the Indiana Board of Public Harbors and Terminals in 1939 which was succeeded by the present Indiana Port Commission in 1961, the relationship of the Mid-West and Bethlehem Steel Corporations to the project, the significance of the project to industry and commerce in general, the investigations of feasibility and preliminary designs made by Sverdrup & Parcel to date, including economic studies, preliminary engineering, and model study of the harbor, and the proposed schedule of construction for various components of the harbor project including the initial grading, roads and railroads, outer breakwater, bulkheads, inner riparian fill and confining structure, dredging, and site facilities to provide an operable harbor by April, 1968.

2. Following a presentation by Pennington of the "Burns Waterway Harbor Preliminary Report of the Rubble Mound Breakwater" prepared by S&P for this particular meeting and dated October 2, 1965, Messrs. Hudson, Bruun and Ayers in turn offered comments as follows:

3. Discussion by Mr. Hudson. In general a conservative design is recommended, using a design wave height of about 16.5 feet. This recommendation is based upon the following considerations:

a. The still water level upon which the design wave is to be superimposed, should be three feet above the Low Water Datum (average lake level 2.0 feet above L.W.D. plus wind set-up of 1.0 feet based on a one month frequency). An increase of wind set-up to 1.5 feet was later suggested. Reference was made to the Gary Harbor design for which the most severe storms from 1929 to 1951 were analyzed. The lake level for this design was set at 2.1 above L.W.D. In 1929, a lake level of plus 6.7 due to wind tide was recorded. The ten year record of Ogden Dunes weather data is considered rather limited for determination of a design wave. Use of winds for ice-free periods only is open to question. It would be preferable to assume that severe winds occurring during the ice season could occur also during the navigation season. A bad storm occurring in early winter might as easily occur a month earlier during the navigation season.

b. A design is recommended based on the modified Beach Erosion Board curve, giving a wave of 18 feet occurring once in 25 years at Station "C" (Chicago). Using a shoaling factor of 0.95 and a refraction factor of 0.95 yields a design wave height, $H_{1/3}$, equal to 18' x 0.95 x 0.95 = 16.2', say 16.5'. The Chicago District Corps of Engineers ran a refraction analysis which resulted in a factor of 1.0, but this was in error because they should have gone to deeper water, which would result in a factor of less than 1.0.

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c. "If I were designing the breakwater, I would use not less than 15. That is doing it the conservative way. With a lake level of 3 feet over L.W.D. plus a 15 foot wave height, or 18 feet above L.W.D., and having for example a crown elevation of only 12 feet, 6 feet of solid water overtopping would result. The crown cannot take this."

d. The establishment of an optimum crown elevation should take into consideration an allowance for some damage. Reference is made to Waterways Experiment Station Research Report No. 2-2 entitled "Design of Quarry Stone, Cover Layers or Rubble Mound Breakwaters", which presents data on laboratory tests to determine percent damage for different values of H. The percent damage figures apply to the percent of the entire trapezoidal cap with base at minus H and crest at plus H. This data cannot be applied directly to a two layer armor system. Laboratory tests have recently been made for a two layer system but results are not as yet available. The design of a crown to withstand overtopping requires specialized know-how.

e. To indicate policy of the Corps of Engineers, a Chicago District design for a breakwater at New Buffalo, Michigan based on a 30 mph design wind was required to be changed by the Division Office to a 40 mph design wind.

f. The cost over a 50 year period should be studied. A conservative job now means less maintenance later.

g. The use of Bedford limestone has disadvantages because of rectangular or cubical shapes and relatively low specific gravity. Stones should be placed with the long axis perpendicular to the slope, but this is feasible only above water level, and creates problems in constructing the underlayer. A one layer system should have units keyed in and at least 35 percent voids, which cannot be obtained with Bedford limestone blocks. A pell-mell arrangement of Bedford stone in one layer is not very stable and is not recommended. It is difficult to make two layers pell-mell with rectangular shapes. Interlocking plus high porosity is the reason for higher coefficients for Tribars or Tetrapods.

h. The Buffalo District of the Corps of Engineers has designed a number of breakwaters using Bedford stone, and apparently do not use generally accepted formulae for design. Failures have probably not occurred due to less severe conditions in Lakes Ontario and Erie.

i. Comments on specific pages of the S&P report follow:

Page 6 - Par. 1 Porosity and angle of wave attack also effect wave run-up.

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Page 7, first line. Would not put much confidence in 0.5 H. Recommend use of 1.0 rather than 1.2.

Page 7, Par. 3. Reference was made to Crescent City and Nawiliwili breakwaters for crown design. The use of concrete posts was suggested to prevent the armor from being thrown upon or over the crest. Crown damage is a problem with any type of armor material in that it is difficult to construct the corners to provide the same degree of keying as obtained on the slope. Would not use one layer of rock for crown.

Page 8, Par. 7. Disagree with second section of paragraph. Believe that the combined effect of velocity and excess hydrostatic pressure is most critical at or near the water line.

Page 9. Value of K = 5.5 should not be used for rough stone except for very special conditions where placement is carefully controlled; 3.5 is acceptable. When overtopping occurs water also flows through the crown. Crescent City tests showed that armor on harbor side should be carried to a point below still water level (10'?) if subject to overtopping.

Page 10, Par. 2d. Tetrapods roll and break when placed on crown.

Page 11, third section of Par. b. One layer of Tribars is almost impossible to lift out. When attacked by waves considerably greater than design wave, failure of the breakwater trunk would be by en-masse sliding. Tetrapods and stone can lift out before reaching the point of en-masse sliding.

Page 12, Par. d. Agree with Palmer.

Page 13, Par. b. K = 5.5 should not be used for two layers of stone. The reasoning leading to K = 2.8 is not correct. "Geometry would run out" (exposing the underlayers).

Page 13, Par. C - Secondary Cover Layer. In discussing this paragraph on composition of various layers, Hudson exhibited Fig. 1 & 2 attached, which he suggested in place of the design indicated in paragraph C. Regarding cross sections in S&P report, the sand layer is up too high where velocities might be destructive. Use of the 1/20 rule for filter design is recommended.

j. With regard to the harbor plan as a whole, Hudson observed that, compared to harbors in general, harbor area is small, making the absorption of wave energy problematical. Once in the harbor, waves will be troublesome, and from this standpoint it would be better to design for no overtopping. Corners are notorious in creating big waves, and small

waves are hard to absorb. The corner should have a gentler slope and should be "beefed" up. At the landward end of the slips, it is desirable to use the 1:4 slopes as proposed at and above the water line to minimize reflection. However, the slope below the minus H level could be steepened to as much as 1:1-1/4. The geometry at the head (entrance) is questioned with regard to diffraction and stability. For navigation purposes it should be curved outward.

4. Discussion by Dr. Bruun:

a. The model study made at the University of Florida Coastal Engineering Laboratory was conducted to determine wave action within the harbor and not for design or stability of the breakwater.

Referring to page 1 of the report there should be a statement made with regard to ice action.

b. Since there are no wave records for the area, "hindcasting" based on wind data has to be relied upon for design of the breakwater.

c. With regard to deep water significant wave heights, a comparison shows that the Darbyshire method used in the United Kingdom gives a maximum wave height of $1.6H_{1/3}$ whereas the Bretschneider method gives $1.78\ H_{1/3}$.

d. A review of wind data indicated considerable discrepancy between wind speeds obtained from the Ogden Dunes and the Chicago Weather Bureau records. Numerous inquiries led to information from the U. S. Weather Bureau in Chicago that a reduction factor should be applied to readings made prior to 1950, due to combined effect of a change in type of wind instruments and level of instruments. There should be a correction of about 20 percent for higher velocities and 15 percent for lower velocities.

e. The Bretschneider method used in conjunction with shallow water coefficients produces a $H_{1/3}$ of 10 or 10.5 feet. The Darbyshire method would produce a lower significant wave.

f. By applying wind instrument corrections, previously hindcast significant wave height reduces considerably.

g. Use of a still water level of plus 3.1' over L.W.D., as recommended by Hudson is agreed to. Raising the inner edge of crown to 12' with the outer edge of crown 11' is suggested.

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h. Reference made to a report entitled "On Optimum Breakwater Design" by J. van de Kreeke and A. Paape. Use of the Hudson Damage Criteria together with the method employed in this report, is recommended to obtain the ideal combination and the "right answer" for design wave height.

i. Using the right wind data and hindcasting procedure, the maximum wave height is 10 feet. A 30 mile per hour wind for 25 hours doesn't really exist.

j. The value of H = 10 feet should permit no damage and no overtopping.

k. With regard to overtopping, using plus 2 ft. as the corrected water table and plus 1 ft. for wind set-up, it is recommended that the height of the harbor-side edge be 13 feet and the lakeside edge 12 feet.

l. Consideration should be given to the use of a wave screen superimposed on top of breakwater. These are used extensively in Scandinavia and Holland. The wave screens are generally built by anchoring 12" x 12" or 10" x 10" vertical timbers into the top of the mound about 3 feet from the seaside edge and anchoring about 4 inch thick planks to these vertical timbers so as to result in a "flash board" type of wall about 3 feet high. A space about 4 inches high should be left along the bottom edge of the screen. Instead of wooden posts, steel rails may be used. After settlement has taken place, the timber wave screen could be replaced by a concrete wall which should be rounded in section on the seaside to permit ice to skid over the top.

m. A crest width of approximately 16 feet is satisfactory. Will take care of overtopping but not over-splash.

n. A concrete cap should not be used. The Dutch use asphalt extensively to "glue" the top together.

o. At Port Skagen, ice forms every winter. Ten metric ton blocks will stand up against wave action but are moved by ice. A one-layer cap system is not recommended as rupture is easily caused by ice action.

p. The most suitable geometry does not use the same slope all the way down. An "S" shaped slope line is recommended with a 1:2 slope (with rock armor) for the upper portion, a berm at the -H level, and possible a slope below the berm 1:1-1/4.

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At this point Hudson commented that "in the Great Lakes they don't seem to worry about ice. If you are worried about ice, contact the Chicago District Corps of Engineers. Mr. Beaudin can tell you whom to contact. Also, Captain Walton ? in Washington, D. C. knows about ice conditions."

q. Use of sand in cross section. (1) Bruun indicated that pumped sand will stand on a slope of about 1:5 to 1:10. To prevent the sand from entering the rock layers consideration might be given to the use of polyvinyl sheets within which the sand is confined. A layer of gravel should be placed between the rock and filter sheet.

(2) Hudson stated that in order to confine sand, graded filter stone must be very carefully placed and believes that it would be preferable not to use sand. The cost of controlling a sand core with filters should be compared to the cost of obtaining low cost rock. In answer to a question regarding availability and cost of quarry run rock, McGavock stated that quarry run rock could be shipped from the Southwest Chicago area by barge and dumped in place at a cost of approximately $5 per ton and that blast-furnace rock (fluxstone) is available from the Mackinac Straits or Drummond Island region. The latter rock could be placed by self-unloading boats at low cost. McGavock pointed out that sand had been used successfully for the Chesapeake Bay Bridge-Tunnel project.

r. Bruun made reference to an article in the "Dock and Harbour Authority" periodical, volume XLV, No. 534, April, 1965. Bruun stated that sand is used extensively, but the placing of filter layers is difficult.

s. Hudson indicated that damage to a breakwater slope takes the shape of a parabola, with the breakwater crest and a point at Elev. -H being the upper and lower limits of damage. The pressure (caused by a combination of velocity and head as water recedes) is greatest at still water level. Bruun indicated that it is very important not to use one layer in this area.

t. Commenting on page 14 of the S&P report, Bruun stated that he agreed with Hudson regarding the construction of graded filters.

u. Bruun continued a discussion of economy and optimum design. He suggested using H = 10 feet with Damage Criteria A, (WES Research Report 2-2, Fig 19 Line AB), i.e., no damage, and H = 12 feet for Damage Criteria B (Line CD). Two layers should be used in the

critical areas, and an "S" shape of the seaside profile is recommended. He referred to a paper he presented at the XVIII International Navigation Congress, Rome, 1953, S.11-Q1, which discusses slopes and up-rush.

v. A discussion on use of Tribars followed, particularly as to what damage criteria should be used. Hudson indicated that for damage criteria for Tribars placed pell-mell in two layers, the same percentage should be used as for rock. A discussion followed with regard to two layers versus one layer of Tribars. Hudson commented: "When you use one layer you don't want any damage." Hudson continued with the discussion of the design of the upper portion of the seaside slope and the crown, and indicated a post at the upper edge of the crown with Tribars immediately below. He suggested calling Palmer for a design of the crown using Tribars on the slopes.

5. Discussion by Mr. Ayers:

a. Criteria for wave height. Ayers referred to Technical Memorandum No. 36 of the Beach Erosion Board which summarizes all available information on lake level and wave height up to the time of publication (1953), including information on statistical probability.

b. A value of $H_{1/3}$ = 12 feet with a period of 7 seconds is recommended, as well as modifying the "blue-book" design as follows (See Figure 3) : - 12 ton rock across top; 10 - 12 ton rock on upper seaside slope; 8 - 10 ton rock below; 10 - 8 ton rock on the harbor-side slope with 10 ton at edge of crown. Lower the mound of sand to 13 foot height. Use crusher run on either side to protect the sand core. The suggested cross section is designed to use the total spectrum of quarry production. Preparation of two designs, one with stone armor and the other with manufactured units is recommended, allowing Contractor to decide which is most economical, rather than taking alternate bids.

c. Investigation of use of a polyvinyl sheet to reduce scour is suggested.

d. There was some discussion about settlement of the foundation due to the weak clay layer below the sand mantle. Rubble mound breakwaters by nature are capable of withstanding settlement without being damaged.

e. Ayers emphasized the importance of protecting the sand, and the use of alternate designs, leaving choice to the Contractor and thus avoiding the problem of unbalanced bids. Bids should be on a lump sum basis.

MEMORANDUM FOR RECORD October 12, 1965

6. To emphasize the reliance which may be placed on model studies, Hudson related an experience with a rubble mound constructed by the Reserve Mining Company. A model study was made to determine the amount of damage that would result if a mound of quarry run rock throughout were built, allowing waves to knock off the top and rebuilding. The model study was verified by actual experience in that the profile remaining after a design wave occurred was the same as the model study had predicted.

7. Further discussion of how to select a design wave height resulted in the following conclusions:

a. Bruun stated that formulae for shallow water waves should be used rather than for deep water waves. Cognizance should be taken of friction and shoaling. Mention was made of 10 feet for no damage and no overtopping and 12 feet for 1 to 5 percent damage.

b. Hudson thought that personally he wouldn't design for less than 14 feet but indicated that the economics of damage versus initial cost should be studied, perhaps beginning with a 10' wave and working upward toward a design which entails a small degree of risk but can be financed. Hudson pointed out that ultimately the Chicago District rather than he will be responsible for approving a design wave height (if such approval is required).

c. Messrs. Kuhn and Green discussed briefly the problem of developing the project with the funds available. It was stated that the breakwater construction, dredging, riparian fill, and initial road and track work, all of which is essential to establishing an initial operable harbor, has to be accomplished for 25½ million dollars. This money is borrowed, in effect, from the State of Indiana and must be paid back.

d. There was some discussion of whether a model study of the cross section was required. Hudson stated that the Corps of Engineers does not run tests except when the waves are breaking on the structure and that if the design as prepared is not too much different from what is generally used, there would be no need for model study. Bruun stated that a model study is not considered necessary for stability (such tests usually requiring at least two months) but may be required for up-rush (this test requiring about 3 to 4 weeks).

e. It was the general consensus that two layer construction should be used on the lakeside and that for nonbreaking waves there was no reason for the two layers to extend more than H below low water.

MEMORANDUM FOR RECORD October 12, 1965

7. Pennington inquired as to whether a 1½:1 slope or 2:1 slope should be used on the lakeside from the standpoint of wave reflection. Bruun recommended using a 2:1 slope (above elev. -H) for a slope protected with rock armor.

8. Mr. McGavock suggested that the next step might be to revise the S&P report on breakwater design and prepare a new report on the rubble mound to confine the riparian fill. It was decided that these reports could be prepared in about two weeks and mailed to the participants of the conference, and that the same group should be brought together for a final discussion sometime around October 24.

H. McGavock
Project Engineer

cc: All Participants

October 29, 1965

MEMORANDUM FOR RECORD

FROM : John H. McGavock

PROJECT : Burns Waterway Harbor

SUBJECT : Conference on Rubble Mound Breakwater and West Outer Bulkhead Design, October 24, 1965

PARTICIPANTS:

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A. INTRODUCTION

Admiral Peltier opened the meeting at 8:30 AM. Mr. McGavock reviewed the meeting held October 3rd, 1965, and asked for comments on the report draft dated October 18, 1965 which had been circulated

to all participants for their review. The purpose of the meeting was to reach decisions on the preliminary designs for the rubble mound breakwater and west outer bulkhead as presented in the two S&P reports of October 2nd and October 18, 1965. Comments were given by Dr. Bruun, Mr. Hudson, and Mr. Ayers and, following this, a general discussion ensued. The meeting adjourned about 3:30 PM.

B. DISCUSSION

The discussions given by Dr. Bruun, Mr. Hudson, and Mr. Ayers, given in chronological order, are given below and, at the end, is a short account of the highlights of a general discussion period.

1. Discussion by Dr. Bruun

a. Wave Height

Reference is made to Figure 5 "Frequency of Occurrence of wave heights in days per year" on Page 18 from S&P's "Burns Waterway Harbor Preliminary Report of Rubble Mound Breakwater and West Outer Bulkhead" dated October 18, 1965. This report is hereinafter referred to as "S&P's Report".

There is a slight uncertainty involved in interpreting this figure. For a 12-ft wave height the expected frequency of occurrence is .04. This means that a 12-ft significant wave will occur one day in 25 years or about 1 hour in one year; in other words, the figure does not (and cannot because of the character of the wind

records from which it was developed) include duration of attack by the given wave height. This uncertainty could add to the determined wave height. When the fetch length becomes greater than 150 miles or so the significant wave height does not increase significantly for a given wind speed. Duration, then becomes the important consideration.

In hindcasting wave height, both Wilson and Bretschneider would take the shape and length of the lake into consideration. This would add a minus to the wave height but involves considerable uncertainty.

b. Material Usage

Since the most economical stone available for Burns Harbor is in the 4 to 12 ton range, it is sound reasoning to combine the W/2 and W layers for economy. Some blocks will be smaller than should be but these could be placed first, saving the larger ones for more critical areas such as near the surface. For an 11-ft wave height, which is a good design wave height, the required armor weight is just 6 tons. There will be many blocks larger than this which will introduce an added safety factor. For instance, 10-ton stones are required to resist a 13-ft significant wave.

c. Ice

Investigation of the ice problem in Alaska indicates that if the ice is caused to move fast (because of tides) then there could be cause for concern. Since there are no tides on Lake Michigan, ice is

not expected to present a problem. This is in agreement with S&P's report of October 18.

d. Slope and Gradation

In reference to Figures 6 and 7 from S&P's report, the slopes of 1.5:1 and 2:1 for Tribar and Rock Armor respectively are good.

As to gradation in the first underlayer, a lighter weight than W/10 could be used below the extent of the main armor layer.

In view of the difficulty of placing good filter layers needed to protect the sand core and since an economical rock core material is available, it is recommended the sand core section be eliminated from consideration.

The W/10 layer shown is more easily constructed flat where it provides a base for the W/2 layer and can be brought up to a higher elevation.

It is good to extend the Tribar armor above the level of the crest height as shown and is also recommended for the section utilizing Rock Armor. This is common practice in the breakwater design in Florida.

e. Breakwater Head and Corner Sections

Consideration must be given to the extreme ends of the breakwater, where tribars are considered superior to tetrapods.

Hudson commented that a combination of head differential, water jet, and gravity are the contributing forces to a breakdown in this region. Tetrapods will tend to roll.

Tribars can be used on the same slope as the trunk section but their weight should be increased.

2. Discussion by Mr. Hudson

a. Slope

It is desirable to flatten the slope of a breakwater for two reasons; (1) a lighter weight rock can be used, maintaining the same degree of stability; although more rock will be required, it could prove cheaper; and (2) the reflected waves are less with a flatter slope, which is desirable for small craft operating near the breakwater.

Mr. Beaudin commented that a considerable number of pleasure craft operate in the area and that they are depending on the Corps of Engineers to assure that their interests are protected. These craft do use the shore line east and west and he felt that for 3 to 4-ft waves there should be considerable wave absorption.

Because of this wave reflection problem a 2:1 slope is recommended (by Hudson) for all armor. Dr. Bruun agrees although it was noted that except for reflection, there is no other reason from an engineering standpoint to go to a 2:1 slope. The reflected wave height will be reduced by about 50% or a little less with a 2:1 slope. There is some difference between reflection coefficients for different types of armor on the same slope but this difference cannot be counted on with only two layers. The coefficient would tend to be lower with Tribar armor (as opposed to rock) because of its high porosity.

b. Wave Height

His original estimate for wave height as presented in the October 3 conference was based on the 1959 Gary Harbor Report which did not include a correction for wind velocity. From this, the maximum deep water wave height is found to be 18 feet. This times .95 refraction coefficient times .95 shoaling coefficient equals a 16.5 foot design wave.

In talks with Dr. Keuligan and Mr. Saville concerning the limited width consideration for the determination of fetch, Dr. Keuligan expressed doubt in the basic physics of Saville's simplified approach to the problem. Saville noted that short-crested waves can go in any direction.

In accordance with Saville's thinking, a 150-mile fetch is obtained for Burns Harbor. The longest straight line length of water, however, is about 300 miles. Averaging these two values gives a design fetch length of 225 miles. Upon entering Bretschneider's 1961 curves with this value and a 35 mph wind velocity, the deep-water design wave is found to be 14.7 ft. This times $.95^2$ to account for refraction and shoaling results in approximately 13-ft for the significant wave height or $H_{1/3}$. This value should be used for stability computations.

It is usually not advisable to use two design wave heights for stability and overtopping considerations. At Burns Harbor,

however, it appears logical to do so from an economic viewpoint. The crown height could be based on an 11 ft wave and if at a later date it appears that overtopping is excessive, a concrete cap could be added.

c. Gradation

With a 13-ft design wave, a unit weight of rock equal to 145 lb/cu ft and placed on a 2:1 slope, the theoretical weight, W, of rock armor required is 10 tons. For gradation purposes .75W to 1.25W could be used, which will provide adequate protection for the larger waves.

A mound with a 2:1 slope using tribar armor has a theoretical armor weight of 3 tons. Four tons, however, is recommended. If a smooth enough underlayer can be obtained, one layer of tribars above the water line placed uniformly would provide good stability. The porosity is still high (about 47% as opposed to 54% for two layers) and they must be lifted up to be displaced. Palmer should be consulted for information on the placing technique with tribars in one or two layers.

d. Breakwater Head

The breakwater head must be strengthened to a greater extent than a normal section. With two layers of tribars forming a conical-shaped head subject to non-breaking waves, the stability coefficient is 7.5 (See Plate 28 in EM 1110-2-2904). The corresponding required weight when placed on a 2:1 slope is 4 tons. Five-ton units.

however, are recommended. Increasing the weight will provide adequate stability without need of fanning the head, which costs considerable extra money.

With respect to stability, there is no need to bend the breakwater lakeward at the end. Bruun commented, however, that this was done because of depth contour and for navigation interests.

e. Cross-section

The primary armor layers should be taken to elevation -13 below low-water datum. At this point a steeper slope should not be introduced until reaching the bottom of the W/2 layer (approx. -1.5H). It would be satisfactory to steepen the slope at -H if the heavy armor weight, W, is carried to a lower elevation.

The W/10 underlayer is for non-throttling purposes and need extend only to just below the primary armor layer level.

A properly designed rubble mound breakwater does not need a concrete cap. If one is used, however, it must be designed to resist uplift pressures. It can usefully serve to increase stability from overtopping and to keep the armor from sliding over the top.

3. Discussion by Mr. Ayers

a. Cross-section

Any of the waves previously discussed are within the realm of providing a good design. Agreement is expressed for use of an 11-ft wave upon which to base the crest height and a 13-ft wave

for stability or armor weight purposes. The added safety factor as presented by Bruun is noteworthy. It is encouraged that the final design be tailored to economy and practicality.

To require a flatter, 2:1, slope requires additional effort on the part of the contractor in placement. There is heavy enough rock available so that it could be placed on a 1.5:1 slope. Effort should be made to utilize the full output of the quarry for maximum economy and ease of placement. If a wider gradation in the armor can be used, higher costs will be avoided.

One layer of tribars above the waterline is discouraged from use from a practical standpoint. Even though stability is theoretically better when placed uniformly, two layers are recommended throughout.

4. General Discussion

Two final questions pertaining to gradation and slope were then raised.

As to gradation, Hudson replied that if you want to use a wide gradation (4 to 12 tons) then you must lower the stability coefficient to say 2.5. Original determination of the stability coefficient for rounded rock averaged 3.2. The average coefficient, however, for a well-graded riprap with wide gradation was found to be 1.5.

Dr. Bruun commented that there is not much possibility of surge as has been experienced in Gary Harbor. Although the possibility

October 29, 1965

of resonance in the revised harbor basin has increased over that in the original layout, it is still to low to cause concern. (See Bruun letter to S&P, dated September 29, 1965).

Mr. Pennington raised a final question as to the economics of going to a 2:1 slope for wave reflection purposes only. Dr. Bruun said that he will write findings to S&P on wave reflection coefficients Upon receiving this information, a better decision can be made pertaining to slope.

Mr. Hudson commented that assuming a 2:1 slope would reduce the wave reflection adequately and incoming waves greater than, say 5 feet (as arbitrarily suggested by Mr. Beaudin) need not be considered. Then you could use the flatter slope in the region from -5 feet below low-water datum to +5 feet above the high still-water level. This, then, would accomplish the reduction in wave reflection desired.

It was generally understood however, that the small boats should not be operating within the limits affected by wave reflection in the first place. Mr. Sawyer commented that small boat warnings are put out in Michigan City Harbor when waves reach about 3 feet in height.

John H. McGavock
Project Engineer

cc: All participants

Appendix 1B
Summary of Damage to Port Facilities and Moored Vessels

INDIANA PORT COMMISSION

Joseph M. Thomas Chairman
Robert M. Schram Vice-Chairman
Wm. E. Shumaker Secretary/Treas.
George A. Nelson Commission Me
William H. Keck Commission Me
Jack P. Fitzgerald Chief Executive and Port Direct
Lewis B. Gratt Deputy Port D

Dr. #13
Burns Harbor
Evaluations

March 9, 1973

Colonel Richard M. Wells
District Engineer
Department of the Army
Chicago District, Corps of Engineers
219 South Dearborn Street
Chicago, Illinois 60604

Re: Elevations of Breakwater at Burns Waterway Harbor

Dear Col. Wells:

Recent observations after a severe storm with north winds on the lake indicate that the breakwater of the Burns Waterway Harbor has lost some stones from its top. We set up an instrument on the north end of the land fill at the harbor and took sightings across the water to the breakwater and it appears that there are areas and small gaps at various points along the breakwater, some of them as much as 3' below the original established level. Also, there was indication that there has been some subsidence of the breakwater.

The breakwater continues to act as a very effective barrier to wave action during observed storms, but we felt that it would be wise to bring this matter to your attention.

Very truly yours,

INDIANA PORT COMMISSION

Tom Bagley.

Tom Bagley, Port Engineer

CTB/sat

Have been in phone contact with
Mr. Bagley: I have made arrangements to
meet and view the structure with Mr. Ba.
I will also have riprap soundings
taken. BAB

PORT OF INDIANA/BURNS WATERWAY HARBOR/P.O. Box 189/Portage, Ind. 46368/(219) 787-8616

SUMMARY OF DAMAGE DOCUMENTATION

DATE	SOURCE/REMARKS
12 March 1970	- Cal-Sag Resident Engineer - Memorandum - No damage to rubblemound was apparent from shore.

(29 Jan 73, H_o=14.1 ft., T_p^*=11.1 sec., SWL=+4.29, rank = 11)

(15 Feb 73, H_o=12.1 ft., T_p^*=10.9 sec., SWL=+4.17, rank = 26)

09 March 1973	- Ltr. Indiana Port Commission to DE - Recent observations after a severe storm have indicated that the breakwater has lost some stones from its top and small gaps have appeared along the breakwater crest with some as much as 3 ft. below the established crest elevation. There is indication that there has been some subsidence.

(18 Mar 73, H_o=12.8 ft., T_p^*=11.1 sec., SWL=+4.35, rank = 20)

1973	- Red Pederson, Kewaunee Field Office - He remembers that a storm in 1973 seemed to produce the first evidence of damage on the breakwater.
09 April 1973	- Chicago District, Corps of Engineers - Solicitation of bids to raise the sunken tender, "Moore", from the southern end of the western slip in the harbor.

(22 Feb 74, H_o=14.4 ft., T_p^*=10.9 sec., SWL=+4.35, rank = 9)

11 October 1974	- Dr. Harris, CERC - Wave Data Collection Programs - The east-west breakwater has suffered extensive damage and needs maintenance. The motion of moored ships and barges is sometimes excessive. The supervising engineer from Bethlehem Steel was assigned to collect information needed to design harbor improvements. Many stones had been dislodged from the breakwater at the north of the harbor. At places the freeboard had been reduced by half of the original value.

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- It was reported that thick ice never forms in the harbor, but that with northerly winds, ice may pile against the breakwater until it spills over the top. Ice may be responsible for much of the breakwater damage.
- It was reported that extremely rough waves in the harbor were quieted within a few minutes last February when ice spilled over the breakwater.

06 February 1975 - Memo from Chief, Operations Div. to Chief, Engineering Div., Chicago District
- There has been considerable settlement and/or dislodgement of individual stones on the Federal breakwater at Burns Waterway Harbor, Indiana. Based on observations made by Indiana Port Commission personnel, large portions of the structure have a top elevation as little as six feet above LWD compared to a design elevation of 14 feet above LWD.

17 March 1975 - Chicago District field trip to investigate conditions
- Mr. Joseph (IPC) stated that the breakwater has been overtopped during the last few years due to a rise in the lake level, such that wave conditions within the harbor are sufficient to break ship mooring lines and in fact 2 large boats have sunk while moored (one being the Corps of Engineers' tug, "Moore"). The Commission's rubble fill retaining structure has been extensively damaged by this wave action.
- Mr. Bagley (IPC) stated he has only been with the Commission since 1973, but has noticed considerable deterioration of the breakwater since, with the process accelerating.
- The breakwater top is below the designed and constructed +14 LWD with numerous gaps down to +6 to +8 LWD allowing overtopping during all seasons. Mr. Bagley also stated that ice has been pushed up and over the breakwater when driven by a north wind.
- As stated by the Commission, the crest of the breakwater appeared considerably below the original +14 ft. crest, with numerous gaps extending to within 2 to 4 feet of the lake level. Sweeping the hand level along the length of breakwater showed that the crest of a major portion was below the elevation of the easternmost end, armored with the

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12-20 ton stone. The reinforced northwest corner is also above most of the north breakwater.

29 April 1975 - Operations Division Memo, Chicago District
- Observations by Operations Division personnel indicate a very minor amount of stone fractures.

18 July 1975 - Memorandum for Record, District Geologist
- Visual inspection of the breakwater indicated that the structure was in far better shape than expected. This undoubtedly is due to some remedial work presently being performed by the Corps of Engineers' Operations on the eastern extreme of the dike.
- The Indiana limestone blocks are in excellent shape and in most cases would only require a minimal amount of remedial armor stone to restore to the original grade lines of elevation +14 ft. LWD.
- In summary, the overall appearance of the stone breakwater (from waterline to crest) does not appear too alarming. The reach where some noticeable crest displacement has occurred is station 43+50 to 46+46. Other low areas within station 0+00 to 18+00 have been replaced with armor stone by Operations Division with good results.

(13 Nov 75, H_o=13.1 ft., T_p^*=10.7 sec., SWL=+3.72, rank = 18)

7 June 1976 - Ltr. Indiana Port Commission to DE, Chicago District
- Asking for information on how much work is to be done on the breakwater during the current season and what sort of overall program the district has in mind for its restoration.
- Stating that the Indiana Port Commission is very interested in the carrying out of measures that will reduce the swell and the wave action within the confines of the harbor.

(01 Feb 1976, H_o=13.8 ft., T_p^*=10.9 sec., SWL=+2.93, rank = 13)

(22 Feb 1976, H_o=15.4 ft., T_p^*=10.9 sec., SWL=+3.44, rank = 6)

29 June 1976 - Ltr. Chicago District DE to IPC
- The current stone placement operations to restore the design elevation of plus 14 feet above Low Water Datum to portions of the breakwater will be completed on 30 June 1976. Further stone placement operations are

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scheduled for later this year in September, subject to the availability of funds.
- It is anticipated that future maintenance of the breakwater will be required annually.

(20 Dec 1976, H_o=11.8 ft., T_p^*=10.7 sec., SWL=+2.25, rank = 30)

11 April 1977
- DF from Chief, Eng. Div. to Chief, Con-Ops Div.
- Regarding a proposed monitoring program at Burns Harbor
- Referenced the 1 October 1976 report "Hydraulic Analysis for Performance of Federal Breakwaters for Burns Harbor, Indiana during period from 1967 to 1975", NCCED-H
- Harbor Response: Preliminary information indicates that wave heights generated in the slips during storms may be dangerous for moored small craft and possibly may interfere with ore unloading operations at the docks. Although the wave conditions in the slips were studied in the University of Florida model, the deep-water input data used in the tests does not agree with current (1975) data supplied by WES.
- The limited data available shows that the wave heights in the harbor generated by wave energy transmission alone through the rubble-mound voids are about 3 feet high at 400 feet from the structure. These waves could increase in height in the vertical-walled slips due to reflections.
- Observed Data: Some measured wave data was recorded during a storm on 13 November 1975. Personnel at the U.S. Coast Guard Station at Michigan City observed wave heights of 12 to 13 feet at 3-hour intervals. Personnel at Burns Harbor observed wave heights of 3 to 6 feet with 4 to 6 second periods during the day. This one event indicates that waves about 12.5 feet high would generate transmitted waves of about 4.5 feet high, which is about twice as high as those predicted.
- An underwater pressure-type was installed at Beverly Shores, Indiana, in 20 feet of water in October 1974. Records for November 1974 through December 1976 show the highest significant wave recorded during this time period occurred in January 1975 with a wave height of 8.5 feet and an associative wave period of 8.5 seconds.

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(14 Jan 79, H_o=13.4 ft., T_p^*=10.9 sec., SWL=+2.30, rank = 16)

(26 Feb 79, H_o=13.8 ft., T_p^*=10.7 sec., SWL=+2.57, rank = 12)

(25 Dec 79, H_o=15.4 ft., T_p^*=11.1 sec., SWL=+3.68, rank = 5)

1980 - Red Pederson, Kewaunee Field Office
- Maintenance activities in 1980 required 3 times the normal amount of maintenance stone up until that time. They found that they had to redo some previous maintenance areas, specifically: the middle 200-300 feet they redid 1977 maintenance work, the northeast point of breakwater they redid 1976 maintenance work, the northwest corner they redid 1976 maintenance work.

14 January 1980 - Ltr. Indiana Port Commission to DE, Chicago District
- A severe storm occurred December 24-25, 1979 with high velocity north-northwest winds.
- As a result, there occurred extensive damage within the harbor in the form of embankment erosion at the north end of the dock in the east harbor arm and the tugboat harbor located in the northeast corner of the riparian fill.

30 January 1980 - Ltr. DE, Chicago District to Indiana Port Commission
- Response to IPC letter dated 14 Jan. 1980.
- Placement of stone along the lakeside has provided increased protection by filling the voids along the breakwater. Upon completion of this work, we will investigate the feasibility of increasing the height of the breakwater to further increase the degree of protection.

(26 Feb 80, H_o=12.1 ft., T_p^*=10.7 sec., SWL=+3.03, rank = 29)

28 May 80 - Annual Harbor Inspection - Kewaunee
- Lakeside of northerly shore connection portion of the north breakwater is in good condition.
- Entire structure should be surveyed to determine settling - underwater/riprap soundings.

(02 Dec 80, H_o=12.8 ft., T_p^*=10.9 sec., SWL=+3.19, rank = 19)

(24 Dec 80, H_o=12.5 ft., T_p^*=10.9 sec., SWL=+2.79, rank = 21)

(20 Nov 81, H_o=12.1 ft., T_p^*=10.9 sec., SWL=+3.08, rank = 28)

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(11 Nov 83, H_o=15.1 ft., T_p^*=11.1 sec., SWL=+4.01, rank = 8)

(16 Nov 83, H_o=12.1 ft., T_p^*=11.1 sec., SWL=+3.25, rank = 26)

(28 Feb 84, H_o=16.4 ft., T_p^*=11.1 sec., SWL=+4.02, rank = 4)

28 February 1984 - Charlie Johnson, NCD, notes
- Storm caused stone damage at Burns Harbor, very large waves inside the harbor, and sinking of a Cargill-chartered barge in the harbor.

2 March 1984 - Ltr. Indiana Port Commission to DE, Chicago District
- The winter storm of February 27-28, 1984 has caused damage to the breakwater. Armor stone has been dislodged in many areas and the overall integrity of the above water stone placement has been affected.

23 March 1984 - DF written by Charlie Johnson, NCD
- Regarding a breakwater inspection
- The breakwater was ice-covered and therefore could not be inspected on the lakeside. The inspection was confined to a long-distance landside view from the west side of the Bethlehem Steel slip.
- Irregularity in crest is visible about 1000 ft westerly from the tip of the north breakwater and it extends for 100-200 ft. along the crest.

(11 Nov 84, H_o=9.5 ft., T_p^*= sec., SWL=+4.20, rank = N/A)

17 December 1984 - Ltr. from Chief, Con-Ops Div. to Indiana Port Commission
- The placement of stone along the harbor breakwater will repair the damage which has been caused by some recent storms. It will also maintain the design height of the breakwater in areas where settlement has been most severe.

18 December 1984 - Ltr. from Indiana Port Commission to DE, Chicago District
- A storm at Burns the weekend of November 10/11, 1984 caused extensive damage to the dock located at the north face of the harbor.

(12 Feb 85, H_o = 17.1 ft., T_p^*=11.1 sec., SWL=+4.35, rank = 3)

3 April 1985 - Field Trip MFR - Jim Knox, District Geologist
- Noted a number of low areas along the breakwater with the principal low area being the

extreme ENE arm of the breakwater for about 400 feet south and west of its end.

- Some cracked, fractured or broken stone was noted but the great bulk of the visible armor stone appears intact and weathering well. For this reason I do not believe the dike has lost height due to stone deterioration but more probably due to mass slumping of the dike. This could be due to armor stone rolling or being rafted by ice incrustations off of the crestal area.
- The armor stone could be upsized. The armor stone here is 10 to 16 ton stone in contrast to the 7 to 20 ton stone at Chicago Harbor and the 10 to 20 ton stone at Milwaukee Harbor which are on the west side of the lake with shorter wind reaches and less exposure to the prevailing westerly winds.
- Another possibility is repair with durable or denser stone less readily affected by weathering or moved by storm waves. A granite from a Wisconsin quarry or a basalt or a anorthosite from northern Michigan would fit this criteria.

September 1985
- NCD memo regarding a sidescan investigation
- Most of the breakwater toe on the lakeward side was armor stone disappearing into the clay. This result was unexpected and is to be further investigated.

30 October 1985
- DF - Emergency stone placement on Burns Harbor Breakwater, Jim Knox, District Geologist
- The middle portion of the breakwater is the most seriously deteriorated.
- Pellmell stone placement with no attempt to interlock the stone has resulted in serious loss of stone and dike height which was as low as 8 feet below the design crest of +14.0 ft. LWD. The Kewaunee stone crew have rebuilt 250 ft. of the breakwater and interlocked the stone on the inside of the breakwater back up to the crest design of +14.0 ft. LWD.

(storm event on 2 Dec 85 did not meet minimum conditions of 9 hours of 20 mph winds out of NW-NE, however, wave gage information indicate the following inside and outside harbor:

Outside: H_s = 3.0 ft.
Inside: H_s = .27 ft.

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2 December 1985	- Field Trip Note, Charlie Johnson, NCD - Transmitted waves caused ship-surge motion at Cargill Dock in Burns Harbor. No damage was done to Burns Harbor Breakwater. Little wave energy was observed over the top of the breakwater. Peak water level was +5.5 ft. LWD at the Calumet Harbor Gage.
2 December 1985	- MFR storm field trip - John Panganiban, NCC, and Charlie Johnson, NCD - A ship moored close to Cargill Dock was observed to have 1.0 m horizontal and 0.5 m vertical movement apparently due to high wind and surge in the harbor. - Through visual observations ..., very little wave energy was transmitted over the break-water. Waves inside the harbor were unusually low steepness, indicating that they must have been transmitted through the break-water. Waves outside the harbor were estimated 10 to 12 feet, 6 to 7 second period.
10 December 1985	- Ltr. Indiana Port Commission to Chief, Operations and Maintenance Branch - ... we wish to express our concern over the ability of the breakwater to effectively dampen the surge and wave action in the harbor area during storm activity from the north. As you are aware, a ship was tied at the northernmost berth at Burns International Harbor. On December 2, 1985, the ship was berthed at the Cargill dock and because of high surge in the harbor repeatedly struck the dock causing hull damage to the ship and flexing the steel sheet piling in the dock causing settle-ment of the paved areas along the dock face.

(08 Feb 87, H_o = 17.7 ft., T_p^*=11.1 sec., SWL=+4.85, rank = 2)

9 February 1987	- Ltr. Indiana Port Commission to DE, Chicago District - On February 8, 1987 a severe storm occurred on Lake Michigan which was accompanied by high winds and waves on the lake. Witnesses have indicated that the waves overtopped the outer breakwater and waves inside the harbor reached heights of 8 ft. or more. - As a result extensive damage was done at the south end of the West Harbor Arm, a pumphouse containing fire protection

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pumping equipment was destroyed, the north face of the undeveloped riparian fill suffered extensive erosion, and the tug boat harbor experienced erosion of the sideslopes to the extent that dredging of this facility will be necessary.

- In addition to the damage done inside of the outer breakwater, the breakwater itself appears to have lost some of the protective armor stone, with more of the loss concentrated on the east end of the structure.

17 June 1987

- MFR - Dive Inspection of Breakwater, Jim Knox, District Geologist
- The light standard was exposed on the harbor face with stone fallen away on this face at 0+00 location. The outer breakwater face appeared good, erosion having occurred chiefly on the inner harbor side slopes.
- The Indiana limestone appeared to be in generally good condition but some spalling was noted in the first 300 feet of the breakwater and there had been separation of stone on bedding planes and stylolities with additional breakage of the resultant elongated slabs.
- The divers surveyed the slope which was supposedly a 1.5 to 1 slope but actually at points was steeper due to stone being piled almost vertically one on top of the other.
- Stone tends to be piled into pyramidal structures with weak interlock between structures that allows individual stone to move down slope. Stone moving down slope dislodges lower stone allowing more stone to move down slope. Stone moving down slope lowers crest. Crest moves lakeward as it lowers and stones move down harbor slopes.
- Recommendations: Raising of structure on harbor side by the width of additional layer of armor stone. Armor stone could be very dense Cedarville dolomite to elev. -5.0 LWD and then brought from there above surface to crest of dike by lighter but better weathering and more resistant to freeze breakage Indiana Limestone. Entire crest of breakwater needs raising and strengthening. Present inner slopes appear to be too steep.

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BURNS HARBOR BREAKWATER
DAMAGE EVALUATION

Damage evaluation for Burns Harbor Break Water was based on observation of several photo sets taking during the period of 17 April 1973 to 17 Mar 1988.

<u>STATION 0+00:</u>
83 - 85: Changed in width at water line between 0+50 - 0+75 on harbor side.
84 - 86: Stones moved from crest to below water line on lake side.
85 - 87: Crest elevation is unchanged
86: Steepening of slope on the lake side between 0+00 - 0+25.

<u>STATION 1+00:</u>
84 - 87: Changed in width at water line between 1+00 - 1+20 on harbor side.
85 - 88: Stones moved from lake side crest to harbor side
85 - 88: Crest elevation between station 1+00 - 2+00 have been changed.
87: Spalls fractures on harbor side from 1+30 - 1+50

<u>STATION 2+00:</u>
85 - 87: Changed in width at water line between 2+50 - 2+75 on harbor side.
85 - 87: Drastic changed on crest elevation between 2+60 - 2+80.
86: Steepening of slope on the harbor side between 2+60 - 2+80.

<u>STATION 3+00:</u>
83 - 87: Changed in width at water line between 3+65 - 3+85 on both sides.
85 - 87: Drastic changed on crest elevation between 3+50 - 3+75.
84 - 87: Stones moved from crest to below water line on the harbor side.
87: Steepening of slope on the lake side between 3+50 - 3+75.

<u>STATION 4+00:</u>
85 - 87: Changed in width at water line from 4+50 - 4+75 on harbor side.
85 - 87: Changed on crest elevation between 4+75 - 5+00
85 - 87: Masses stones moved from crest to below water line on harbor side.

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STATION 5+00:

85 - 87: Changed in width at water line from 4+90 - 5+10 on harbor side.
85 - 87: Changed on crest elevation between 4+90 - 5+50
85 - 87: Masses stones moved from crest to below water line on harbor side.

STATION 6+00:

84 - 86: Changed in width at water line and slope on both side of the break water.
84 - 87: Damage occurred at waterline on harbor side
85 - 87: Stones moved from crest to below water line on the harbor side between 6+50 - 5+50.

STATION 7+00:

83: Narrow section from 7+50 - 8+00 at water line on both sides.
84: Narrow section from 7+50 - 8+00 at water line on both sides.
87: Low crest at 7+50
88: Low crest at 7+50

STATION 8+00:

83: Changed in width at water line from 7+50 - 8+00 on both sides.

STATION 9+00:

85 - 88: Some stone lost between Sept '85 and July'88. Large stone present at crest in 1985 photo but absent from 1988 photo. Could not tell which direction stone fell.

STATION 10+00:

83 - 88: Narrow section at water line on both sides.
85 - 88: Low at crest.
85 - 88: Hole at station 10+00 in '85 and still present in 1988.

STATION 11+00:

83 - 87: Narrow section at water line

STATION 12+00:

83: Steepening of slope on the lake side between 11+75-12+15
84 - 87: Changed in width at water line between 12+00 - 12+50 on lake side.

STATION 13+00:

87: Narrow section from 12+75 - 13+45 on the lake side.

STATION 14+00:
85 - 87: Changed in width at water line from 13+75 - 14+20 on the harbor side.
85 - 87: Changed on crest elevation from 13+75 -14+10

STATION 15+00:
83 - 84: Some stones lost on the lake side
83 - 84: Crest elevation is unchanged.
87: Narrow section from 14+90 - 15+80 on the lake side.

STATION 16+00:
83 - 84 Few stones lost on the lake side
83 - 87: Changed in width at water line from 16+00 - 16+40 on the harbor side.

STATION 17+00:
83 - 87: Narrow section at water line from 17+20 - 18+00 on harbor side.
87: Steepening of slope from 17+50 - 18+00 on harbor side

STATION 18+00:
83 - 88: No change on both side.

STATION 19+00:
83 - 86: No change on both side.

STATION 20+00:
83 - 87: Few stones lost at 20+00 - 20+35 on harbor side.

STATION 21+00:
83: Crest elevation is low at 20+85 - 21+15
83 - 87: Changed in width at water line from 21+30 - 21+60 on the harbor side.

STATION 22+00:
84 - 87: Changed on crest elevation at 21+80 - 22+45

STATION 23+00:
83: Narrow section at water line on both side.
83: Steepening of slope from 22+55 -22+90 and 23+00 - 23+60.

STATION 24+00:
83 - 85: Few stones lost at 24+75 on lake side.

STATION 25+00:
83 - 85: Stones moved upslope on crest and lake side
83 - 85: Few stones lost at 25+30 on lake side

STATION 26+00:
83 - 85: Few stones lost at 26+75 - 26+85 on crest.

STATION 27+00:

83 - 85: Stones moved downslope toward toe on harbor side
83 - 85: Few stones rotated, kicked landward on lake side.
83 - 86: Stone on top at 26+75 ('83 photo) gone by '86

STATION 28+00:

83 - 84: Few stones lost on lake side
83 - 86: Stones moved upslope on the lake side between 28+50 - 29+00

STATION 29+00:

83 - 85: Stones lost and damage between 29+00 - 30+00 on harbor side.
83 - 85: Few stones rotated, kicked landward on harbor side

STATION 30+00:

83 - 85: Stones lost and damage between 30+00 - 30+50 on harbor side.

STATION 31+00:

83 - 87: Narrow section at water line between 30+75 - 31+40 on the harbor side.

STATION 32+00:

83 - 87: Changed in width at water line from 32+50 - 33+00 on the lake side.

STATION 33+00:

83 - 86: Stones moved downslope from crest to below water and some damage on harbor side from 33+00 - 34+50
83 - 87: Changed in width at water line from 33+00 - 33+50 on the lake side.

STATION 34+00:

84 - 85: Changed on crest elevation from 34+20 - 34+50
84 - 85: Damaged on harbor side

STATION 35+00:

83 - 86: Changed in width at water line from 35+30 - 35+65 on the lake side.
85 - 86: Some stones lost on the harbor side
85 - 87: Steepening of slope at 35+00 on harbor side.
85 - 88: Stones moved downslope toward harbor from 35+10 - 35+70.

STATION 36+00:

83: Steepening of slope from 36+00 - 36+50 on the lake side.
86: Narrow section at water line on lake side from 36+00 - 36+50.

STATION 37+00:

84 - 85: One stone on crest at 37+60 moved upslope toward harbor from lake side.

87: Narrow section at water line on lake side from 37+00 - 38+00.

STATION 38+00:

84 - 85: Stones moved landward toward harbor from crest.

86: Narrow section on lake side from 38+50 - 39+00

STATION 39+00:

87: Narrow section on both sides from 39+80 - 40+00

STATION 40+00:

84 - 87: Changed in width at water line from 40+00 - 40+75 on the harbor side.

STATION 41+00:

84 - 86: Changed in width at water line from 41+35 - 41+75 on the lake side.

84 - 86: Few stones lost at 41+65 on the lake side

STATION 42+00:

84 - 86: Changed in width at water line from 42+15 - 42+70 on the lake side.

84 - 86: Few stones lost at 42+30 - 42+50 on the lake side

STATION 43+00:

86: Narrow section on lake side from 43+35 - 43+85

STATION 44+00:

84 - 85: Some stones moved down slope on harbor side.

86: Narrow section on harbor side at 44+00

STATION 45+00:

86: Narrow section at water line on harbor side from 44+85 - 45+20

86: Steepening of slope from 44+85 - 45+20 on harbor side.

STATION 46+00:

88 - 89: Light tower fell landward.
Very difficult to see from existing photos.

BURNS HARBOR BREAKWATER
DAMAGE EVALUATION

DAMAGE MAY HAVE BEEN CAUSED BY **11 NOV 1983** STORM EVENT

STATION 0+00:
83 - 85: Changed in width at water line between 0+50 - 0+75 on harbor side.
83 - 86: Stones moved from crest to below water line on lake side.
86: Steepening of slope on the lake side between 0+00 - 0+25.

STATION 2+00:
86: Steepening of slope on the harbor side between 2+60 - 2+80.

STATION 3+00:
83 - 87: Changed in width at water line between 3+65 - 3+85 on both sides.
87: Steepening of slope on the lake side between 3+50 - 3+75.

STATION 5+00:
83 - 87: Changed in width at water line from 4+90 - 5+10 on harbor side.

STATION 7+00:
83: Narrow section from 7+50 - 8+00 at water line on both sides.
87: Low crest at 7+50

STATION 8+00:
83: Changed in width at water line from 7+50 - 8+00 on both sides.

STATION 10+00:
83 - 88: Narrow section at water line on both sides.

STATION 11+00:
83 - 87: Narrow section at water line

STATION 12+00:
83: Steepening of slope on the lake side between 11+75-12+15
84 - 87: Changed in width at water line between 12+00 - 12+50 on lake side.

STATION 13+00:
87: Narrow section from 12+75 - 13+45 on the lake side.

STATION 15+00:
83 - 84: Some stones lost on the lake side
83 - 84: Crest elevation is unchanged.
87: Narrow section from 14+90 - 15+80 on the lake side.

STATION 16+00:
83 - 84: Few stones lost on the lake side
83 - 87: Changed in width at water line from 16+00 - 16+40 on the harbor side.

STATION 17+00:
83 - 87: Narrow section at water line from 17+20 - 18+00 on harbor side.
87: Steepening of slope from 17+50 -18+00 on harbor side.

STATION 20+00:
83 - 87: Few stones lost at 20+00 - 20+35 on harbor side.

STATION 21+00:
83: Crest elevation is low at 20+85 - 21+15
83 - 87: Changed in width at water line from 21+30 - 21+60 on the harbor side.

STATION 23+00:
83: Narrow section at water line on both side.
83: Steepening of slope from 22+55 -22+90 and 23+00 - 23+60.

STATION 24+00:
83 - 85: Few stones lost at 24+75 on lake side.

STATION 25+00:
83 - 85: Stones moved upslope on crest and lake side
83 - 85: Few stones lost at 25+30 on lake side

STATION 26+00:
83 - 85: Few stones lost at 26+75 - 26+85 on crest.

STATION 27+00:
83 - 85: Stones moved downslope toward toe on harbor side
83 - 85: Few stones rotated, kicked landward on lake side.
83 - 86: Stone on top at 26+75 ('83 photo) gone by '86

STATION 28+00:
83 - 84: Few stones lost on lake side
83 - 86: Stones moved upslope on the lake side between 28+50 - 29+00

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STATION 29+00:

83 - 85: Stones lost and damage between 29+00 - 30+00 on harbor side.

83 - 85: Few stones rotated, kicked landward on harbor side

STATION 30+00:

83 - 85: Stones lost and damage between 30+00 - 30+50 on harbor side.

STATION 31+00:

83 - 87: Narrow section at water line between 30+75 - 31+40 on the harbor side.

STATION 32+00:

83 - 87: Changed in width at water line from 32+50 - 33+00 on the lake side.

STATION 33+00:

83 - 86: Stones moved downslope from crest to below water and some damage on harbor side from 33+00 - 34+50

83 - 87: Changed in width at water line from 33+00 - 33+50 on the lake side.

STATION 35+00:

83 - 86: Changed in width at water line from 35+30 - 35+65 on the lake side.

STATION 36+00:

83: Steepening of slope from 36+00 - 36+50 on the lake side.

STATION 37+00:

87: Narrow section at water line on lake side from 37+00 - 38+00.

STATION 43+00:

86: Narrow section on lake side from 43+35 - 43+85

STATION 44+00:

86: Narrow section on harbor side at 44+00

STATION 45+00:

86: Narrow section at water line on harbor side from 44+85 - 45+20

86: Steepening of slope from 44+85 - 45+20 on harbor side.

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BURNS HARBOR BREAKWATER
DAMAGE EVALUATION

DAMAGE MAY HAE BEEN CAUSED BY **28 FEB 1984** STORM EVENT

STATION 0+00:
83 - 85: Changed in width at water line between 0+50 - 0+75 on harbor side.
84 - 86: Stones moved from crest to below water line on lake side.
86: Steepening of slope on the lake side between 0+00 - 0+25.

STATION 1+00:
84 - 87: Changed in width at water line between 1+00 - 1+20 on harbor side.

STATION 2+00:
86: Steepening of slope on the harbor side between 2+60 - 2+80.

STATION 3+00:
83 - 87: Changed in width at water line between 3+65 - 3+85 on both sides.
84 - 87: Stones moved from crest to below water line on the harbor side.
87: Steepening of slope on the lake side between 3+50 - 3+75.

STATION 6+00:
84 - 86: Changed in width at water line and slope on both side of the break water.
84 - 87: Damage occurred at waterline on harbor side

STATION 7+00:
84: Narrow section from 7+50 - 8+00 at water line on both sides..
87: Low crest at 7+50

STATION 10+00:
83 - 88: Narrow section at water line on both sides

STATION 11+00:
83 - 87: Narrow section at water line

STATION 12+00:
84 - 87: Changed in width at water line between 12+00 - 12+50 on lake side.

STATION 13+00:
87: Narrow section from 12+75 - 13+45 on the lake side.

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STATION 15+00:
83 - 84: Some stones lost on the lake side
83 - 84: Crest elevation is unchanged.
87: Narrow section from 14+90 - 15+80 on the lake side.

STATION 16+00:
83 - 84 Few stones lost on the lake side
83 - 87: Changed in width at water line from 16+00 - 16+40 on the harbor side.

STATION 17+00:
83 - 87: Narrow section at water line from 17+20 - 18+00 on harbor side.
87: Steepening of slope from 17+50 -18+00 on harbor side

STATION 20+00:
83 - 87: Few stones lost at 20+00 - 20+35 on harbor side.

STATION 21+00:
83 - 87: Changed in width at water line from 21+30 - 21+60 on the harbor side.

STATION 22+00:
84 - 87: Changed on crest elevation at 21+80 - 22+45

STATION 24+00:
83 - 85: Few stones lost at 24+75 on lake side.

STATION 25+00:
83 - 85: Stones moved upslope on crest and lake side
83 - 85: Few stones lost at 25+30 on lake side

STATION 26+00:
83 - 85: Few stones lost at 26+75 - 26+85 on crest.

STATION 27+00:
83 - 85: Stones moved downslope toward toe on harbor side
83 - 85: Few stones rotated, kicked landward on lake side.
83 - 86: Stone on top at 26+75 ('83 photo) gone by '86

STATION 28+00:
83 - 84: Few stones lost on lake side
83 - 86: Stones moved upslope on the lake side between 28+50 - 29+00

STATION 29+00:
83 - 85: Stones lost and damage between 29+00 - 30+00 on harbor side.
83 - 85: Few stones rotated, kicked landward on harbor side

STATION 30+00:
83 - 85: Stones lost and damage between 30+00 - 30+50 on harbor side.

STATION 31+00:
83 - 87: Narrow section at water line between 30+75 - 31+40 on the harbor side.

STATION 32+00:
83 - 87: Changed in width at water line from 32+50 - 33+00 on the lake side.

STATION 33+00:
83 - 86: Stones moved downslope from crest to below water and some damage on harbor side from 33+00 - 34+50
83 - 87: Changed in width at water line from 33+00 - 33+50 on the lake side.

STATION 34+00:
84 - 85: Changed on crest elevation from 34+20 - 34+50
84 - 85: Damaged on harbor side

STATION 35+00:
83 - 86: Changed in width at water line from 35+30 - 35+65 on the lake side.

STATION 36+00:
86: Narrow section at water line on lake side from 36+00 - 36+50.

STATION 37+00:
84 - 85: One stone on crest at 37+60 moved upslope toward harbor from lake side.
87: Narrow section at water line on lake side from 37+00 - 38+00.

STATION 38+00:
84 - 85: Stones moved landward toward harbor from crest.
86: Narrow section on lake side from 38+50 - 39+00

STATION 39+00:
87: Narrow section on both sides from 39+80 - 40+00

STATION 40+00:
84 - 87: Changed in width at water line from 40+00 - 40+75 on the harbor side.

STATION 41+00:
84 - 86: Changed in width at water line from 41+35 - 41+75 on the lake side.
84 - 86: Few stones lost at 41+65 on the lake side

STATION 42+00:

84 - 86: Changed in width at water line from 42+15 - 42+70 on the lake side.

84 - 86: Few stones lost at 42+30 - 42+50 on the lake side

STATION 43+00:

86: Narrow section on lake side from 43+35 - 43+85

STATION 44+00:

84 - 85: Some stones moved down slope on harbor side.

86: Narrow section on harbor side at 44+00

STATION 45+00:

86: Narrow section at water line on harbor side from 44+85 - 45+20

86: Steepening of slope from 44+85 - 45+20 on harbor side.

BURNS HARBOR BREAKWATER
DAMAGE EVALUATION

DAMAGE MAY HAVE BEEN CAUSE BY **12 FEB 1985** STORM EVENT

STATION 0+00:
83 - 85: Changed in width at water line between 0+50 - 0+75 on harbor side.
84 - 86: Stones moved from crest to below water line on lake side.
86: Steepening of slope on the lake side between 0+00 - 0+25.

STATION 1+00:
84 - 87: Changed in width at water line between 1+00 - 1+20 on harbor side.

STATION 2+00:
86: Steepening of slope on the harbor side between 2+60 - 2+80.

STATION 3+00:
83 - 87: Changed in width at water line between 3+65 - 3+85 on both sides.
84 - 87: Stones moved from crest to below water line on the harbor side.
87: Steepening of slope on the lake side between 3+50 - 3+75.

STATION 6+00:
84 - 86: Changed in width at water line and slope on both side of the break water.
84 - 87: Damage occurred at waterline on harbor side

STATION 7+00:
87: Low crest at 7+50

STATION 10+00:
83 - 88: Narrow section at water line on both sides.

STATION 11+00:
83 - 87: Narrow section at water line

STATION 12+00:
84 - 87: Changed in width at water line between 12+00 - 12+50 on lake side.

STATION 13+00:
87: Narrow section from 12+75 - 13+45 on the lake side.

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STATION 15+00:

87: Narrow section from 14+90 - 15+80 on the lake side.

STATION 16+00:

83 - 87: Changed in width at water line from 16+00 - 16+40 on the harbor side.

STATION 17+00:

83 - 87: Narrow section at water line from 17+20 - 18+00 on harbor side.

87: Steepening of slope from 17+50 -18+00 on harbor side

STATION 20+00:

83 - 87: Few stones lost at 20+00 - 20+35 on harbor side.

STATION 21+00:

83 - 87: Changed in width at water line from 21+30 - 21+60 on the harbor side.

STATION 22+00:

84 - 87: Changed on crest elevation at 21+80 - 22+45

STATION 24+00:

83 - 85: Few stones lost at 24+75 on lake side.

STATION 25+00:

83 - 85: Stones moved upslope on crest and lake side

83 - 85: Few stones lost at 25+30 on lake side

STATION 26+00:

83 - 85: Few stones lost at 26+75 - 26+85 on crest.

STATION 27+00:

83 - 85: Stones moved downslope toward toe on harbor side

83 - 85: Few stones rotated, kicked landward on lake side.

83 - 86: Stone on top at 26+75 ('83 photo) gone by '86

STATION 28+00:

83 - 86: Stones moved upslope on the lake side between 28+50 - 29+00

STATION 29+00:

83 - 85: Stones lost and damage between 29+00 - 30+00 on harbor side.

83 - 85: Few stones rotated, kicked landward on harbor side

STATION 30+00:

83 - 85: Stones lost and damage between 30+00 - 30+50 on harbor side.

STATION 31+00:

83 - 87: Narrow section at water line between 30+75 - 31+40 on the harbor side.

STATION 32+00:

83 - 87: Changed in width at water line from 32+50 - 33+00 on the lake side.

STATION 33+00:

83 - 86: Stones moved downslope from crest to below water and some damage on harbor side from 33+00 - 34+50

83 - 87: Changed in width at water line from 33+00 - 33+50 on the lake side.

STATION 34+00:

84 - 85: Changed on crest elevation from 34+20 - 34+50

84 - 85: Damaged on harbor side

STATION 35+00:

83 - 86: Changed in width at water line from 35+30 - 35+65 on the lake side.

STATION 36+00:

86: Narrow section at water line on lake side from 36+00 - 36+50.

STATION 37+00:

84 - 85: One stone on crest at 37+60 moved upslope toward harbor from lake side.

87: Narrow section at water line on lake side from 37+00 - 38+00.

STATION 38+00:

84 - 85: Stones moved landward toward harbor from crest.

86: Narrow section on lake side from 38+50 - 39+00

STATION 39+00:

87: Narrow section on both sides from 39+80 - 40+00

STATION 40+00:

84 - 87: Changed in width at water line from 40+00 - 40+75 on the harbor side.

STATION 41+00:

84 - 86: Changed in width at water line from 41+35 - 41+75 on the lake side.

84 - 86: Few stones lost at 41+65 on the lake side

STATION 42+00:

84 - 86: Changed in width at water line from 42+15 - 42+70 on the lake side.

84 - 86: Few stones lost at 42+30 - 42+50 on the lake side

STATION 43+00:

86: Narrow section on lake side from 43+35 - 43+85

STATION 44+00:

84 - 85: Some stones moved down slope on harbor side.

86: Narrow section on harbor side at 44+00

STATION 45+00:

86: Narrow section at water line on harbor side from 44+85 - 45+20

86: Steepening of slope from 44+85 - 45+20 on harbor side.

BURNS HARBOR BREAKWATER
DAMAGE EVALUATION

DAMAGE MAY HAVE BEEN CAUSED BY 08 FEB 1987 STORM EVENT

STATION 1+00:

84 - 87: Changed in width at water line between 1+00 - 1+20 on harbor side.
85 - 88: Stones moved from lake side crest to harbor side crest.
85 - 88: Crest elevation between station 1+00 - 2+00 have been changed.

STATION 2+00:

85 - 87: Changed in width at water line between 2+50 - 2+75 on harbor side.
85 - 87: Drastic changed on crest elevation between 2+60 - 2+80.

STATION 3+00:

83 - 87: Changed in width at water line between 3+65 - 3+85 on both sides.
85 - 87: Drastic changed on crest elevation between 3+50 - 3+75.
87: Steepening of slope on the lake side between 3+50 - 3+75.
84 - 87: Stones moved from crest to below water line on the harbor side.

STATION 4+00:

85 - 87: Changed in width at water line from 4+50 - 4+75 on harbor side.
85 - 87: Changed on crest elevation between 4+75 - 5+00
85 - 87: Masses stones moved from crest to below water line on harbor side.

STATION 5+00:

85 - 87: Changed in width at water line from 4+90 - 5+10 on harbor side.
85 - 87: Changed on crest elevation between 4+90 - 5+50
85 - 87: Masses stones moved from crest to below water line on harbor side.

STATION 6+00:

84 - 87: Damage occurred at waterline on harbor side
85 - 87: Stones moved from crest to below water line on the harbor side between 6+50 - 5+50.

STATION 7+00:

88: Low crest at 7+50

STATION 9+00:

85 - 88: Some stone lost between Sept '85 and July'88. Large stone present at crest in 1985 photo but absent from 1988 photo. Could not tell which direction stone fell.

STATION 10+00:

83 - 88: Narrow section at water line on both sides.
85 - 88: Low at crest.
85 - 88: Hole at station 10+00 in '85 and still present in 1988.

STATION 11+00:

83 - 87: Narrow section at water line

STATION 12+00:

84 - 87: Changed in width at water line between 12+00 - 12+50 on lake side.

STATION 13+00:

87: Narrow section from 12+75 - 13+45 on the lake side.

STATION 14+00:

85 - 87: Changed in width at water line from 13+75 - 14+20 on the harbor side.
85 - 87: Changed on crest elevation from 13+75 -14+10

STATION 15+00:

87: Narrow section from 14+90 - 15+80 on the lake side.

STATION 16+00:

83 - 87: Changed in width at water line from 16+00 - 16+40 on the harbor side.

STATION 17+00:

83 - 87: Narrow section at water line from 17+20 - 18+00 on harbor side.
87: Steepening of slope from 17+50 -18+00 on harbor side

STATION 20+00:

83 - 87: Few stones lost at 20+00 - 20+35 on harbor side.

STATION 21+00:

83 - 87: Changed in width at water line from 21+30 - 21+60 on the harbor side.

STATION 22+00:

84 - 87: Changed on crest elevation at 21+80 - 22+45

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STATION 31+00:

83 - 87: Narrow section at water line between 30+75 - 31+40 on the harbor side.

STATION 32+00:

83 - 87: Changed in width at water line from 32+50 - 33+00 on the lake side.

STATION 33+00:

83 - 87: Changed in width at water line from 33+00 - 33+50 on the lake side.

STATION 35+00:

85 - 88: Stones moved downslope toward harbor from 35+10 - 35+70.

85 - 87: Steepening of slope at 35+00 on harbor side.

STATION 37+00:

87: Narrow section at water line on lake side from 37+00 - 38+00.

STATION 39+00:

87: Narrow section on both sides from 39+80 - 40+00

STATION 40+00:

84 - 87: Changed in width at water line from 40+00 - 40+75 on the harbor side.

CARGILL
GRAIN
DIVISION

6600 Highway 12
Port of Indiana
Portage, IN 46368-1286

BIH

MAY 19, 1992

TO: U.S. ARMY CORP OF ENGINEERS

FROM: CARGILL INC, BURNS INTERNATIONAL HARBOR

SUBJECT: INFORMATION REQUESTED

Cargill Inc, began it's grain operation at Burns International Harbor in May of 1981. Since that time, three major incidents have occurred at Cargill which are directly traceable to Lake Michigan storms at BIH. The dates of these incidents are: February 28, 1984, February 8, 1987 and February 15, 1991. At this time I would like to briefly report on each incident.

1. February 28, 1984

   Due to a severe Lake Michigan storm on February 27 and February 28, two barges loaded with winter wheat sank. The VL-7781 contained 46,958 bushels and the RMT-320 contained 49,325 bushels.

   Settlement on the claims for cost incurred was $568,000.

   There was no interruption of business.

2. February 8, 1987

   Damage to the Cargill dock and shoreline occurred because of the Lake Michigan storm of February 7 and February 8. Approximately 65 feet of shoreline from each side of the dock was eroded away. Major sinking of the dock surface also occurred. New rip-rap had to be installed along with fill and surface work to the dock. Cost of the damage-Port of Indiana would have this figure.

   There was no major interruption of business.

3. February 15, 1991

On February 14 and February 15 there was a severe Lake Michigan storm resulting in the sinking of barge DM-1866. This barge was loaded with 47,669 bushels of soybeans. Total damage was approximately $375,000.

There was no interruption of business.

In all three incidents there was no interruption of business because of either timing of the year or location of the sunken barges. However, if the barges would have sank directly in front of the load out dock or if in 1987 we would have been in a major load out program, the interruption of business would have been very costly.

Sincerely,.

Bob Fifield

Bob Fifield, Location Manager

INDIANA PORT COMMISSION BURNS INTERNATIONAL HARBOR • 6600 U.S. HIGHWAY 12 PORTAGE, INDIANA 46368 • USA • (219) 787-8636 • FAX (219) 787-8842

July 20, 1992

Mr. David Wallin
Department of the Army
Corps of Engineers
Chicago District
111 North Canal Street
Chicago, IL 60606-7206

Re: Burns International Harbor
Breakwater Evaluation

Dear Mr. Wallin:

Occasional winter storms from 1968 thru present have produced waves in the harbor at Burns International Harbor of sufficient magnitude to inflict damage on the Outer Harbor Rubble Mound. This letter attempts to generally identify the storm event and to established a dollar amount of damage incurred by the Commission as a result of each event. Events where damage was experienced will be listed from the most recent and work back in time. The date of the damage will be listed, a brief description, and the best available cost for repairs. The events are as follows:

1. Storm, February 14, 1991: During said storm event a barge broke loose from its moorings at the Cargill Dock and damaged a walkway leading from the shoreline to a mooring cell situated approximately 200' east of the Cargill Dock, and two (2) pipe pile fender dolphins that flank the Cargill Dock to be damaged. The damage inflicted to the walkway was of a nature whereby it was repairable, however the fender dolphins had to be replaced. The cost for repairing and resetting the walkway was $16,105.00, and the cost to repair the fendering dolphins was $50,600.00, for a total cost of $66,705.00.

2. Storm, February 8, 1987: This storm event inflicted the greatest amount of damage on facilities at Burns International Harbor than any previous or subsequent storm. Nearly all of the rubble mound on the north face of the riparian fill was completely destroyed; the work boat harbor's side slopes were severely eroded and the harbor had to be dredged to re-establish depth; the riprapped slopes on either side of the dock, South End West Harbor Arm were severely eroded and had to be rebuilt; and extensive damage was done to the fire protection pump house by waves which is also situated at the South End of West Harbor. The aggregate cost to repair all of the above described damage was $1,012,251.40.

"FOREIGN TRADE ZONE #152"

Letter, Mr. David Wallin
July 20, 1992
Page 2

3. Storm, November, 1984: This storm event produced wave energy that caused a ship moored at the Cargill Dock, Outer Harbor, to damage said dock structure. The damages were destruction of several sheet piles of the main bulkhead wall on both the corners of the dock. These piles ruptured and resulted in the loss of substantial amounts of backfill from behind the wall as well. In addition extensive damage was inflicted on the concrete gantry shiploader rail system. New concrete rail beams and foundations had to be installed at both ends of the craneway. The cost of these repairs to the Commission was $75,233.44

4. Storm, February 28, 1984: This storm developed waves within the harbor that caused severe damage to portions of the rubble mound along the north face of the riparian fill. The damages were concentrated on the area west of the Cargill Dock with the most severe occurring on the northwest corner. In addition, the riprapped slope at the South End of the West Harbor Arm was also heavily damaged. The cost to repair these damages was $91,731.70.

5. Storm, December 24, 1979: This storm produced waves that resulted in damage to the north face of the rubble mound and damage to the work boat harbor. Cost data on the damage is difficult to ascertain because a contract specific for repairs was not done. Repairs were implemented by Contractors that had equipment on site at the time, and armor material for the rubble mound was not purchased because some material had been stockpiled, and large blocks of waste concrete were obtained at little cost to the Commission to fortify the rubble mound. It is estimated that the aggregate cost for these repairs was approximately $30,000.

6. Storms, Jan., Feb., March, 1973: From past records, there appears to have been a series of storms in early 1973 that caused damage to the north face of the rubble mound. From incomplete records, it appears that material for the repairs was purchased at a cost of $11,900 and the repairs were done for $20,800, for a total cost of $32,700.00

7. Storms, Early 1970: We have record of a contract to reconstruct the rubble mound in early 1970. It is difficult to point to any particular storm event that caused such excessive damage to the rubble structure, however [illegible] reconst[illegible]ction cost from old contract records was $169,950.

INDIANA PORT COMMISSION
BURNS INTERNATIONAL HARBOR CLARK MARITIME CENTRE SOUTHWIND MA[illegible]E CENTRE

Letter, Mr. David Wallin
July 20, 1992
Page 3

Cost Summary:

1.	Storm, 1991	$ 66,705.00
2.	Storm, 1987	1,012,251.40
3.	Storm, 1984 - Nov.	75,233.44
4.	Storm, 1984 - Feb.	91,731.40
5.	Storm, 1979 - Dec.	30,000.00
6.	Storms, Early 1973	32,700.00
7.	Storms, Early 1970	169,950.00
	Total Approximate Damages	$1,478,571.54

Please keep in mind that the cost data for the damage for 1979 back is estimated and may not be entirely accurate. A thorough review of the accounts would be necessary if the exact amounts are required. However, the information provided as to the dates and damages sustained are believed to be relatively accurate.

In response to your other question of acceptable wave heights within the harbor, it is appropriate to strive to keep the wave in the harbor to a level that would not place a loaded barge in peril. Typically the freeboard on a loaded barge is 1.5' to 2.0' out of the water. Waves higher than this that would break over the barge deck and threaten to swamp a barge are undesirable.

I hope that information will assist the Corps with the study of the Breakwater at Burns International Harbor that is currently underway. If you have any further questions or need any additional information please feel free to contact me.

Best Regards
INDIANA PORT COMMISSION

John W. Hughes

John W. Hughes, P.E.
Director of Engineering

JWH/pc
cc:
Frank G. Martin, Jr.
James Hartung
Pete McCarthy
Bill Fritchley

INDIANA PORT COMMISSION
BURNS INTERNATIONAL HARBOR CLARK MARITIME CENTRE SOUTHWIND MARITIME CENTRE

Appendix 1C
Burns Harbor, Indiana, Nomination for Inclusion in MCCP Program

DEPARTMENT OF THE ARMY
NORTH CENTRAL DIVISION, CORPS OF ENGINEERS
536 SOUTH CLARK STREET
CHICAGO, ILLINOIS 60605

NCDED-C

13 JUN 1984

SUBJECT: Monitoring Completed Coastal Projects (MCCP)

Cdr, USACE (DAEN-CWH-H)
WASH DC 20314

1. Reference, (a) DAEN-CWH-H 30 Mar 1984 multiple letter SAB to coastal Division offices, suspense 25 May 1984. Suspense date slipped by permission of J. Lockhart, DAEN-CWH-D, (b) NCDED 16 April 1984 multiple letter to NCD coastal Districts, SAB, and District responses thereto, copies attached.

2. We assign first priority to Burns Harbor, Indiana, for inclusion in the subject program. This harbor is a deep-draft facility with rubble breakwaters extending to depths exceeding 40 feet below low-water datum. The breakwater cross-section was flume-tested at WES in 1967; those tests are described in CERC SR-5, Coastal Hydraulic Models, May 1979, pp. 370-378. The breakwater is a classic two-layer random-placement multigraded stone rubble-mound structure. Foundation conditions were adverse, requiring excavation of as much as about 15 feet vertically of soft clay from the lakebed, then sand backfill, prior to building the mound. The model tests showed that the armorstone had very high stability coefficients against non-breaking wave attack (K_D=8 to 14). Formal completion of the general navigation structures was in October 1968.

3. Since completion, the local sponsor has complained frequently about excessive wave heights and loss of crest elevation. Attached is a long series of correspondence beginning in March 1973 and continuing to March 1984 summarizing these complaints and the Corps responses. To the original 230,000 tons of 10-20 ton armor stone has been added 106,000 additional tons to keep the crest at grade. Damage patterns appear more consistent with settlement than with wave-induced stone motion. Repair efforts have not concentrated on maintaining the subaqueous slope of the mound; note the continuing suggestion for cross-sectional surveys in the condition-inspection reports. The last such survey was done in 1975.

4. Burns Harbor is thus of special interest because geotechnical problems may contribute to the overall problem. The breakwater was also designed for nonbreaking wave conditions with stone sizes which may have very little reserve stability in the local storm-wave climate. Wave transmission through the armorstone layers appears to be objectionable. We have inclosed a copy of the original geotechnical analysis for this structure, along with a representative cross-section of the breakwater and excavation, plus a photograph of the foundation excavation in progress.

5. Subsurface seismic profiling of the sand/soft-clay interface would form a major part of the monitoring effort. We would look for evidence of deformation, inadequate excavation, or faulting along this surface. Side-scan sonar coverage of the breakwater faces and toe areas would be a second major part of the effort.

We would look for whatever armor stones may have rolled down the breakwater face. A third major part of the effort would be conventional soundings of the lakebed on both sides of the breakwater and profiles of the breakwater itself every 100 feet along the centerline. The fourth and final major part would measurement of wave heights inside and outside the breakwater. We would use self-contained recording pressure-type wave gages, which would be left in the water during winter because major storms often occur at the end of open-water periods in winter.

6. Some portions of the proposed monitoring effort would interface nicely with another such effort near the Burns Waterway Sec. 107 Harbor about one-half mile westerly from the deep-draft harbor. Monitoring of sand movement and beach erosion downdrift of the smallboat harbor has been mandated as part of a consent decree to forestall attempts by local shoreline residents to seek a temperary injunction against construction-contract award. Some mobilization and instrumentation costs might be shared between the two efforts. We will commence monitoring this shoreline later on in FY 84.

7. The magnitude of the Burns Harbor effort is comparable to that for Cleveland Harbor. A very rough estimate for the cost would be $150,000 - $200,000 overall.

8. The features proposed to be covered at Burns Harbor address some of the concerns mentioned by Detroit District about two-layer pell-mell stone placement. It is traditional in Operations and Maintenance to seek to repair any and all stone displacements as quickly as possible. This tradition originated because the standard Great Lakes breakwater for almost a century before 1965 had been single-layer laid-up placement. Such breakwaters are widespread in the Lakes, unquestionably successful, and unquestionably constructible. However, prompt replacement of any displaced armorstones on such structures is essential. Burns Harbor was the first two-layer pell-mell placement breakwater built in the Great Lakes, although two-layer stone armoring on crib faces had been done earlier at a few sites. Experience at Burns, as described above and in the inclosures, has left a lingering suspicion that the two-layer pell-mell placement philosophy is inherently deficient, both stability-wise and permeability-wise. New Buffalo Harbor, MI., was subsequently built in 1975 about 25 miles NE from Burns Harbor with single-layer laid-up placed stone because of these suspicions. The failures cited in the NCE letter, while pointing out the lack of confidence in two-layer construction, do not necessarily indicate inadequate stability. The Charlevoix North Pier Sec. B rehab was designed so that waves broke directly on underlayer stone. The Muskegon failure was confined to the only rehabbed section which did not have the old concrete cap curbings placed along the toe; failure took place after a very heavy December rainstorm and windstorm which may have caused toe scour. Damage at Saginaw Bay Confined Disposal Facility took place in spring 1983 and is being repaired with 1100 tons of stone. Like Burns Harbor, this disposal facility is not attacked by breaking waves; it was designed for a KD of 4.0 whereas the Burns Harbor stable KD for limited overtopping was shown to be 8. If the latter KD is correct for blasted stone as well as cut stone, wave damage at the disposal facility would have required 8 ft. waves. Single-layer laid-up placement structures are not immune from stone slippage. The New Buffalo

Harbor main breakwater required repair in 1983 due to settlement, stone slippage, and uneven stone hardness. As another example, the single-layer breakwater at Knife River, MN., was initially built with 5-ton stone, levelled to the waterline during the first storm season, then rebuilt with 10-ton max. stone, and has since been quite stable. The Ludington breakwater failure was a single-layer structure and may have been a timber-crib collapse due to ice thrust.

9. We agree fully with Detroit District's statements regarding the success of single-layer armor. We strongly recommend that Detroit District be granted funds to prepare a report describing the design, examples construction, and maintenance histories of all this type of structure on Lakes Huron, Michigan, and Superior. Estimated cost would be about one engineering man-year, $45,000, conservatively.

10. Especially in shallow water, two-layer stone breakwaters are extremely permeable. This permeability allows considerable sand transport and wave action into the structures. Lexington Harbor, MI. has very large stones, designed for breaking wave conditions, and also has very permeable stone placement. A very simple monitoring effort consisting of rod soundings and probings along the breakwater axis would detect sand in the breakwater voids and would tell us whether sand known to be entering the harbor is doing so through the breakwater. Cost of such an effort to describe sediment accumulation inside the harbor, which would otherwise be based on analysis of existing data, would be estimated at $15,000.

11. The Buffalo District nomination describes unexpected phenomena at Lake Shore Park, Ashtabula, Ohio, which are similar to some at Evanston, IL. In both cases fine sand is moved subaqueously past a revetment from an area of greater wave exposure to one of less exposure, even if the entire nearby shoreline is already somewhat sheltered. The as-placed beach sand was very fine and well-sorted. Rapid subaqueous movement of fine sand under wave attack is very common along all Great Lakes shorelines. At Lake Shore Park, it appears that the offshore breakwaters would have to have higher freeboards to retain fine sand or, alternatively, a considerably coarser-graded sand would have to be used on the beach. We suggest that this project could be inexpensively monitored on a site-visit basis. When a coarser-graded sand is installed, such a monitoring effort would illustrate the important qualitative differences in offshore breakwater function on fine-sand and coarse-sand beaches, respectively.

12. All of the nominations described above have considerable merit. Many of them can be monitored and reported on at low cost. We suggest that you consider limiting each Division's MCCP dollar amounts instead of limiting the numbers of projects monitored in each Division. We also wish to point out that littoral transport processes continue to be important areas of concern in all harbor design and maintenance efforts, in spite of the low priority assigned to reporting experience therewith. The 1977 and 1984 (draft) Shore Protection Manual chapters on littoral processes are almost entirely inconsistent with Corps experience on the Great Lakes. In the absence of a formal reporting process for recording this experience, we must pass it along locally by site visit and informal

communication. This is a very tenuous way of preserving knowledge. Just as the very substantial coastal engineering skills of our pre-World War I predecessors has been largely lost, we fear that our experiences in littoral processes may also become lost.

FOR THE COMMANDER:

6 Incl
as

Zane M. Goodwin

ZANE M. GOODWIN, P.E.
Chief, Engineering Division

CF: w/o incl:

NCBED-DC
NCCPE-HH
NCECO-MO

DEPARTMENT OF THE ARMY
CHICAGO DISTRICT, CORPS OF ENGINEERS
219 SOUTH DEARBORN STREET
CHICAGO, ILLINOIS 60604

NCCPE-HH

15 MAY 1984

SUBJECT: Monitoring Completed Coastal Projects (MCCP)

Commander, North Central Division
ATTN: NCDED-C

1. Reference NCDED-C 16 April 84 letter, SAB.

2. IAW your request, a proposed monitoring program for gathering coastal wave data and present foundation information for Burns Waterway Harbor, Indiana is provided. Reference materials cited in the fact sheet are available upon request. For additional information contact Mr. Harry Krampitz at 353-6474.

Incl
as

CHRISTOS A. DOVAS, P.E.
LTC, Corps of Engineers
Commanding

NCCPE-HH

May 1984

PROPOSED COASTAL MONITORING PROGRAM AT BURNS WATERWAY HARBOR

1. PURPOSE. The purpose of this report is to provide pertinent facts for a Coastal Monitoring Program at Burns Waterway Harbor, Indiana.

2. PROJECT BACKGROUND. The project was authorized by 1965 River and Harbor Act (H.Doc.160, 88th Congress 1st Session) Burns Harbor completed in 1970, was constructed as a joint venture between Bethlehem Steel Corporation (BSC), Mid West Steel Corporation and the State of Indiana. The Corps of Engineers was first involved in the project when a Hydraulic Model Study conducted for the State of Indiana under the direction of Sverdrup and Parcel and Associates, Inc. performed by the College of Engineering, University of Florida was reviewed by the Corps Chicago District in 1966. The harbor consists of a 5830 foot long rubble north breakwater connected to the shore on the west, and riprap and steel bulkheads around the interior (Figure 1). The harbor is maintained at project depths of between 27 and 30 feet. The North breakwater has a modern multi-layered random-placement stone cross-section with the toe at about -40 ft. LWD. It is exposed to over-water fetches exceeding 250 miles. The north breakwater, the one most exposed to wave action, has suffered extensive damage, possibly due to ice. The motion of moored ships and barges in the harbor is sometimes excessive. Further information can be found in references 3b, 3f, 3g, and 3j below.

3. REFERENCES

a. Interim Report on Burns Waterway Harbor, Indiana, Chicago District Corps of Engineers, January 1962

b. Hydraulic Model Study Burns Waterway Harbor for the State of Indiana by Sverdrup and Parcel and Associates, Inc., St. Louis, Missouri, March 1966 (Appendix D Burns Waterway Harbor Design Memorandum)

c. Burns Harbor Model Test Breakwater Design Correspondence File 1968

d. Burns Harbor, Indiana Hydraulic Analysis for Performance of Federal Breakwaters for Period 1967 to 1975

e. 1975 Riprap Soundings, Burns Waterway Harbor, Indiana North Breakwater U.S.A.E.D. Corps of Engineers, Chicago, Illinois File No. ((60.6-R8)/1)

f. Final Report (1977) Burns Harbor, Wave Study by Dr. Dinorah Esteva COB CEAC

g. Wave Data on Burns Waterway Harbor General Background Information 1977

h. Report on Indiana Shore Erosion, Vol II, Preliminary Feasibility Report, Reaches 2-5, Chicago District, May 1978

i. Burns Waterway Small Boat Harbor Final Detailed Project Report Main Report and Final Environmental Impact Statement, Feb. 1983

j. Annual Structure Inspection-Breakwater - Rubblemound 30 June 1983, Burns Waterway North Breakwater File Chicago District Construction Division Maintenance Branch

k. NCDED-C Letter dated 16 April 1984 Subject: "Monitoring Completed Coastal Projects."

4. OBJECTIVES: The objectives of the monitoring program are (1) to document and evaluate the wave action outside the harbor, in the entrance channel and in the harbor at the mooring docks, (2) to provide up to date information on littoral transport from the easterly and westerly directions, and (3) to provide as built information on the foundation conditions of the North Breakwater by appropriate exploratory techniques (4) to provide information for future project designs.

5. Summary of Available Design Report

Reference 3b reviewed the design proposed by Sverdrup and Parcel and Associates by using a Hydraulic model study which considered wind and wave characteristics, 2 types of long waves namely the storm surge and seiche and their occurrence.

The model used flutter type waves and surge generated waves of the flutter type. The model recommended a breakwater tip length of 700 feet, a 10° orientation for the North breakwater, a rounded jetty head and no change from the slopes recommended in the design suggested by Sverdrup and Parcel and Associates. The study also recommended that the rock revetment on the outer slope be arranged in such a manner as to display maximum degree of permeability. Littoral deposits in the entrance area were expected to be small since the depth of 30 to 40 feet was thought to provide little probability for interference with longshore drifts.

The drift was found to be westerly. Indirect comparitive studies were used to predict that the flared entrance would completely prevent the bypassing of littoral drift around the North Breakwater because of the 40 foot depth. Erosion on the downdrift west beach was believed to be possible as soon as the harbor was constructed. Ice jams were predicted to be caused by the North Breakwater's tendency to catch ice.

6. Results of Previous Corps Post Construction Studies

A. Performance Evaluation. The purpose of the study reported below as reference 3d was to review the responses of the North and West Federal, (Burns Waterway Harbor) rubblemound breakwaters under natural conditions during the 8 year period, 1967 to 1975. Results of the 1967 WES Model Study for stability of the structures was also reviewed and remedial measures were recommended.

Reference 3d reported that the average annual damage to the North Breakwater Lakeside armor units is 2.2 times the predicted damage of 0.64 percent. The average annual damage to the west breakwater lakeside armor units is the same as

the predicted damage. The average annual loss of north breakwater cover areas was 0.57 percent and the average annual gain of the west breakwater cover areas was 0.23 percent.

B. Burns Harbor Michigan City Wave Study. Reference 3f reports the available results obtained from a wave study initiated in 1974 which was terminated after 8 weeks of intermittent data was gathered from 18 November 77 to 9 January 78. The data was intermittent due to a lightning storm and intermittent power failures causing two of the gages to malfunction. Reference 3f concluded that:

Conditions leading to extreme or excessive wave activity in the harbor were not recorded during the period of data collection.

The available data indicate that long period wind waves are more predominant in the West slip, while in the East slip it is the shorter period wind waves that dominate the spectrum. Longer period harbor resonance is also indicated in the West slip. However, no conclusive evidence was obtained to quantify the magnitude or frequency of occurrence of harbor wave problems over the long term.

Refraction diagrams prepared by WES (Memorandum for Record - Resio, 17 September 1975) indicate waves from the NW cause larger concentration of wave energy at the North breakwater than those from the NE. Longer period waves from the North refract only at the harbor mouth. Thus wave conditions in the West slip may be expected to be greatly influenced by transmission of longer period waves through the North breakwater while the East slip will be more influenced by diffraction through the harbor mouth.

Greater damage to the North breakwater is to be expected when winds from the North and NW are experienced. However, during the data collection period in this study, the highest waves reached only 7.7 feet at the Michigan City site and one and two feet at the West and East slips, respectively.

C. Annual Structure Inspections and Stone Placement. In 1980 and 1983 structure inspections were conducted along the North Breakwater, Reference 3j. Uneven differential settlement was noted in 1980 and reported to be from station 18 to station 28 in 1983. The armor layer had voids in 1980 but was reported to be in good condition in 1983. Underwater failure was not visible and misalignment was not noted in either report. Maintenance placed 16,730 tons of stone in 1975, 17,266 tons in 1976, 10,026 tons in 1977, 14,340.25 tons in 1978, 0 tons in 1979, and 47,334.255 tons in 1980. If the stone is not replaced overtopping conditions become more severe. The current maintenance objectives is to maintain the original design elevation. — 6,957 TONS IN 1982

7. Description of Problem

A. Littoral Transport. Reference 3h indicates the average net littoral transport rate at Burns Ditch is 20,500 cubic yards per year to the west, while reference 3a states that the net westward transport at Burns Waterway Harbor is 27,000 cubic yards annually. Construction at the westerly Midwest steel Bulkhead in 1966, Burns Waterway Harbor from 1967 through 1970 and the Bethlehem

Steel bulkhead and fill area cut off the littoral supply to reach 3 from the east, while Gary Harbor and the U.S. Steel Corporation land fill dike at the western edge of this reach have cut off the littoral supply from the west. Comparison of aerial photos taken prior to construction of the Burns Waterway Harbor complex with recent photos, indicates a large accretion fillet east of the Harbor complex with significant erosion of riparian lands immediately westward of the mouth of Burns Ditch. Lake levels have been higher than average since initiation of construction of the harbor complex which has intensified the erosion rates. The recent accretion and erosion confirms the anticipated impacts of the harbor complex construction, with the reach immediately west of the mouth of the Burns Ditch mouth being a littorally starved beach subject to significant erosion.

B. Wave Data. The Chicago District conducted a performance evaluation study, reference 3d, analyzing breakwater damage data for period 1967 to 1975.

In reference 3d, it was stated that:

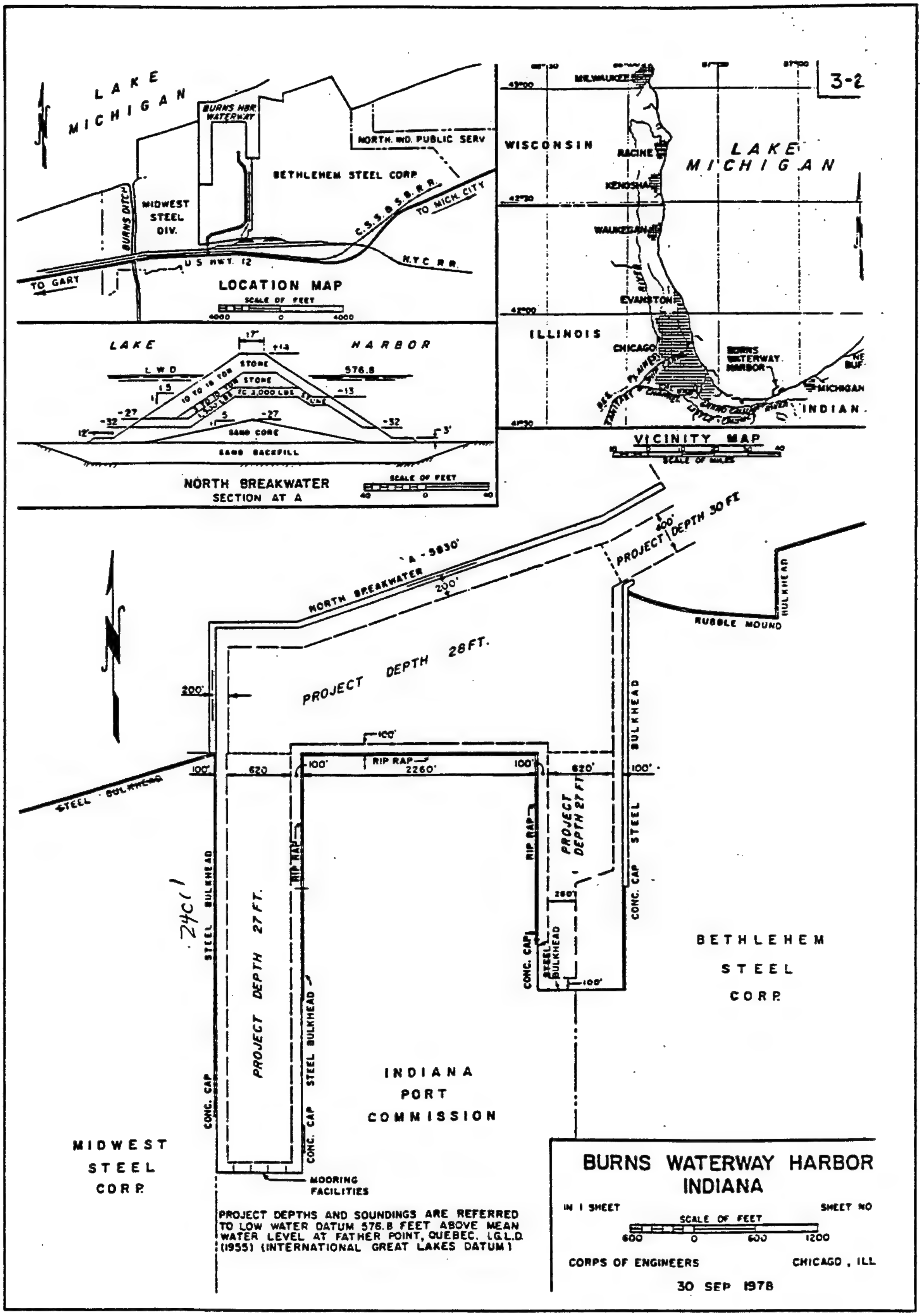
Harbor responses. Preliminary information indicates that wave heights generated in the slips during storms may be dangerous for moored small craft and possibly may interfere with ore unloading operations at the docks. Although the wave conditions in the slips were studied in the University of Florida model, the deep-water input data used in the tests does not agree with current data supplied by WES. The limited data in reference 3d shows that the wave heights in the harbor generated by wave energy transmission alone through the rubble-mound voids are about 3 feet high at 400 feet from the structure. These waves could increase in height in the vertical-walled slips due to reflections. If sufficient data obtained from the wave gages shows that excessive wave heights develop in the harbor and if investigation shows that difficulties to navigation or unloading operations at the docks are experienced, this would suggest that another model study be conducted to determine the extent of the problem and to recommend corrective measures.

Observed data. Some measured wave data was recorded during a storm on 13 November 1975. Personnel at the U.S. Coast Guard Station at Michigan City observed wave heights of 12 to 13 feet at 3-hour intervals. Personnel at Burns Harbor observed wave heights of 3 to 6 feet with 4 to 6 sec periods during the day. This one event indicates that waves about 12.5 feet high would generate transmitted waves about 4.5 ft. high, which is about twice as high as those predicted.

C. Structural Stability. Reference 3k refers to the foundation aspects of the North breakwater at Burns Waterway Harbor. A picture from Reference 3j (FIG-2) shows a dip in the section. This could possibly be the result of a pocket of soft clay underlaying the section and wave setup overloading the structure from the top with the force being transmitted to the foundation horizontally and vertically pushing stone out in the harbor in a bulge detected by the soundings.

8. CONCLUSIONS. It appears that a study of wave data and present foundation conditions are necessary at Burns Waterway Harbor. Visual surveys were performed in 1980 and 1983 of the structural condition of the North breakwater (see paragraph 6c. this report). Localized settlement was visually observed in those

inspections. As stated in reference 3k the problem is complex. Close coordination between the Hydraulics Branch and the Geotechnical Branch and CERC and the WES Geotechnical Lab will be needed to provide a workable solution to the problems at Burns Waterway Harbor. Part of this effort should include an extensive wave data survey and foundation investigation to include appropriate seismic and other exploratory techniques. CERC has the capability to do seismic exploration (see CERC Miscellaneous Report No 79-3 Sand Resources of Southeastern Lake Michigan). This capability should be utilized if feasible to determine present wave and foundation conditions.


3-2
LAKE MICHIGAN
BURNS HBR WATERWAY
NORTH. IND. PUBLIC SERV
BETHLEHEM STEEL CORP
MIDWEST STEEL DIV.
BURNS DITCH
TO MICH. CITY
C.S.S. & S.B. R.R.
U S HWY. 12
N.Y.C. R.R.
TO GARY
LOCATION MAP
SCALE OF FEET
4000 0 4000
LAKE HARBOR
L.W.D. 576.8
STONE
SAND CORE
SAND BACKFILL
NORTH BREAKWATER
SECTION AT A
SCALE OF FEET
40 0 40
WISCONSIN
MILWAUKEE
RACINE
KENOSHA
WAUKEGAN
EVANSTON
ILLINOIS
CHICAGO
BURNS WATERWAY HARBOR
MICHIGAN
INDIANA
VICINITY MAP
SCALE OF MILES
NORTH BREAKWATER
PROJECT DEPTH 30 FT
PROJECT DEPTH 28 FT.
BULKHEAD
RUBBLE MOUND
STEEL BULKHEAD
RIP RAP
2260'
PROJECT DEPTH 27 FT.
STEEL BULKHEAD
CONC. CAP
MOORING FACILITIES
INDIANA PORT COMMISSION
BETHLEHEM STEEL CORP.
MIDWEST STEEL CORP.
24c(1)
PROJECT DEPTHS AND SOUNDINGS ARE REFERRED TO LOW WATER DATUM 576.8 FEET ABOVE MEAN WATER LEVEL AT FATHER POINT, QUEBEC. I.G.L.D. (1955) (INTERNATIONAL GREAT LAKES DATUM)
BURNS WATERWAY HARBOR
INDIANA
IN 1 SHEET
SHEET NO
SCALE OF FEET
600 0 600 1200
CORPS OF ENGINEERS
CHICAGO, ILL.
30 SEP 1978

No.3 BURNS HARBOR 30 JUNE 1983

North breakwater, Reach A, sta.18+00, lake side, showing low section.

No.4 BURNS HARBOR 30 JUNE 1983

North breakwater, Reach A, sta.20+00, harbor side, stationing painted.

FIGURE-2

S: 15 May 1984

~~JOHNSON~~/st/4-11/3-6347

HILPAKKA

PAREZ

~~LEONARD~~

GOODWIN

NCDED-C

16 APR 1984

SUBJECT: Monitoring Completed Coastal Projects (MCCP)

Commander, Buffalo District
Commander, Chicago District
Commander, Detroit District

1. Reference DAEN-CWH-D 30 March 84 letter, SAB, copy attached.

2. The MCCP program is conducted under O&M funding and is administered through DAEN-CWH in cooperation with CERC. District personnel are expected to actively participate in the planning, execution, and analysis of each monitoring effort. We consider this program important because it recognizes that experience with existing projects is one of the best guides to design of new structures and repair of older ones.

3. Examples of projects which we believe are excellent candidates for monitoring include Burns Harbor, Indiana; Lexington Harbor, Michigan; Little Lake Harbor, Michigan; and possibly, to the extent that they are not already being monitored, some of our confined disposal facilities.

4. Burns Harbor was completed in 1970. The breakwater has a modern multilayered random-placement stone cross-section with the toe at about -40 ft. LWD. It is exposed to over-water fetches exceeding 250 miles. The cross-section was flume-tested at WES, where stability coefficients for 2-layer random-placement cut stone were found to range from 8.1 (no overtopping) to 14.0 (high water, considerable overtopping). Foundation conditions were poor. Considerable quantities of soft clay were excavated and replaced with sand to form a stabler foundation. About 230,000 tons of 10-20 ton armor stone were used in the armor layer. Upon completion of construction, maintenance equipment and funding was used to place an additional 60,000 tons of smaller stone within the armor layer to reduce void sizes. The breakwater requires frequent repair, usually attributed to foundation instability. It can be seen that this structure has many features and problems not encountered with the Cleveland Harbor structure.

5. Lexington Harbor, built in 1975, is another harbor with generously sized random-placement armor stone and large voids within the armor layer. Large shoals have developed inside the breakwater which beneficially damp waves transmitted through the voids but which also threaten to interfere with the use of the navigation channel. Perhaps a case could be made for chinking the voids. Also, a large cast-in-place walkway on the breakwater crest is beginning to be

NCDED-C
SUBJECT: Monitoring Completed Coastal Projects (MCCP)

moved relative to the armor stones.

6. Little Lake Harbor is an excellent example of a severe shoaling problem caused by an overlapping breakwater dogleg on a sandy shore. It appears that this problem might be substantially eased if gravel and cobbles accreting at the west side of the harbor were to be placed on the fine sand beach east of it, reducing the availability of fine sand to be carried into the harbor entry. If such placement were to be made, Little Lake would be a splendid demonstration of the importance of nearshore particle sizes in harbor-shoaling mechanisms, hence a very worthwhile subject for monitoring.

7. Confined disposal facilities have presented many difficult design problems in foundation design, runup estimation, overtopping-volume estimation, permeability, supernatant drainage, and constructability. Many of these structures receive limited monitoring to ensure that pollutants do not escape, but it would appear desirable to monitor such structures from a coastal design standpoint.

8. You are free to nominate other projects, old or new. Nominations should include site maps, photographs, a clear discussion of the objectives of the proposed effort, and a brief discussion of what measurements need to be made. Nominations should reach this office by 15 May 84.

FOR THE COMMANDER:

1 Incl
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KANE M. GOODWIN, P.E.
Chief, Engineering Division

2 Results of Analysis of Wave Measurements at Burns Harbor

by David D. McGehee and Joon P. Rhee[1]

Introduction

The U.S. Army Engineer District, Chicago (NCC) is responsible for the operation and maintenance of the Burns Harbor breakwater, located on the southern end of Lake Michigan (Figure 2-1). Burns Harbor was selected for study by the Monitoring Completed Coastal Projects Program (MCCP), a U.S. Army Corps of Engineers research program directed to evaluate design practices and reduce maintenance costs. MCCP is managed by the U.S. Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC). This study, including the wave measurements, was conducted by the Prototype Measurement and Analysis Branch (PMAB) of CERC.

When nominated by NCC for inclusion under MCCP, two specific complaints were cited: the recurring need for repair of the armor layer, and excessive wave action in the harbor, specifically at the Cargill grain loading dock. Identifying the nature, cause, and solution of these problems depends upon knowledge of the wave field outside and inside the harbor. Thus, a wave measurement effort was one of the major components of the monitoring plan for Burns Harbor breakwater.

Wave measurements were obtained at intermittent intervals from gages deployed at four locations inside and outside the harbor between December 1985 and June 1988. In addition, an oceanographic buoy (Station 45007) operated by the National Data Buoy Center (NDBC) since 1981 provides

[1] Research Hydraulic Engineers, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

meteorological and nondirectional wave measurements in deep water (525 ft (160 m)) approximately 65 n.m. (120 km) north of the harbor (Figure 2-1). Figure 2-2 shows the locations of the gages, and Figure 2-3 provides the data availability for the five locations. Monthly plots of reduced wave parameters for each site are provided in Appendix 2A.

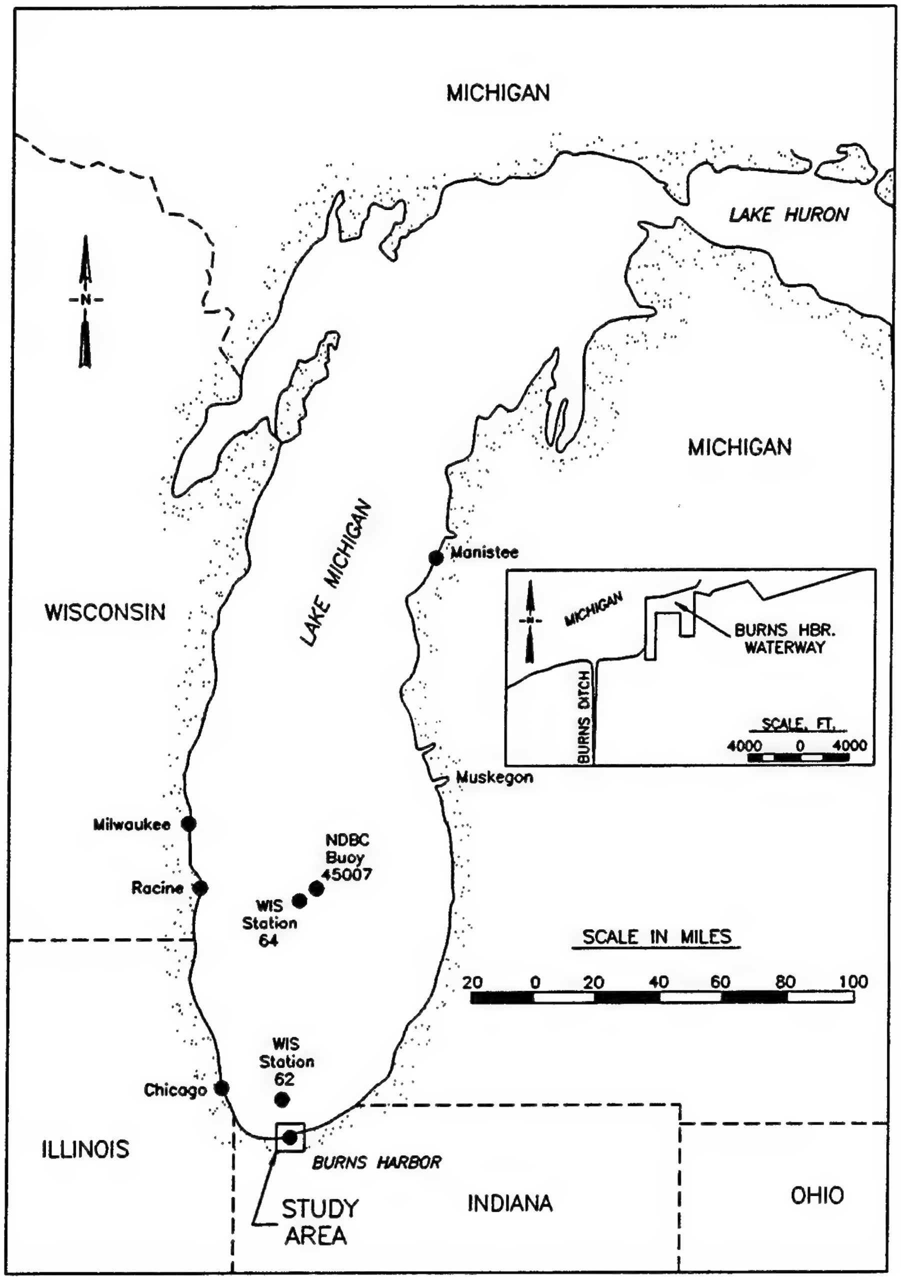

Figure 2-1. Burns Harbor, IN, vicinity map and site map

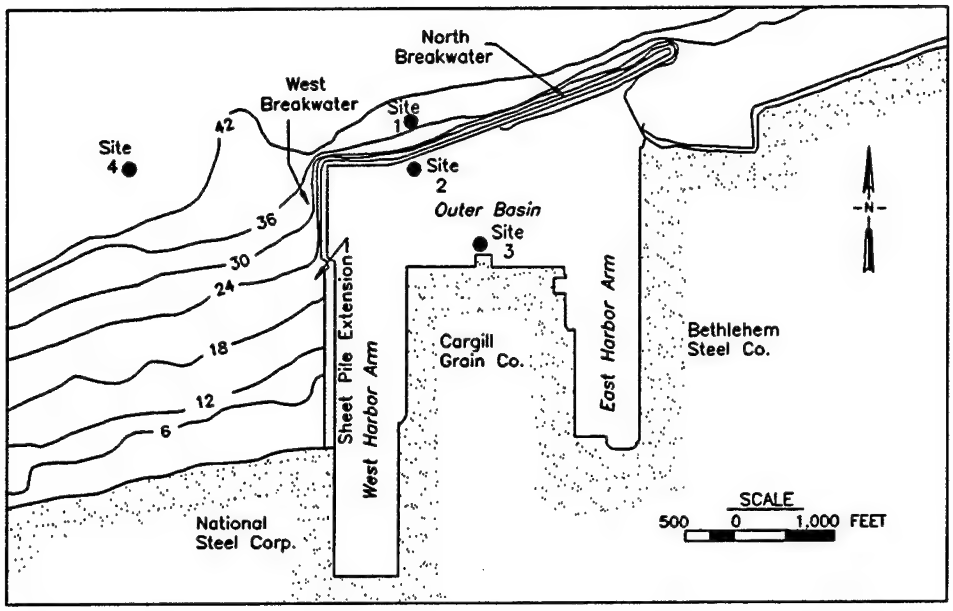

Figure 2-2. Location of wave gages in vicinity of Burns Harbor

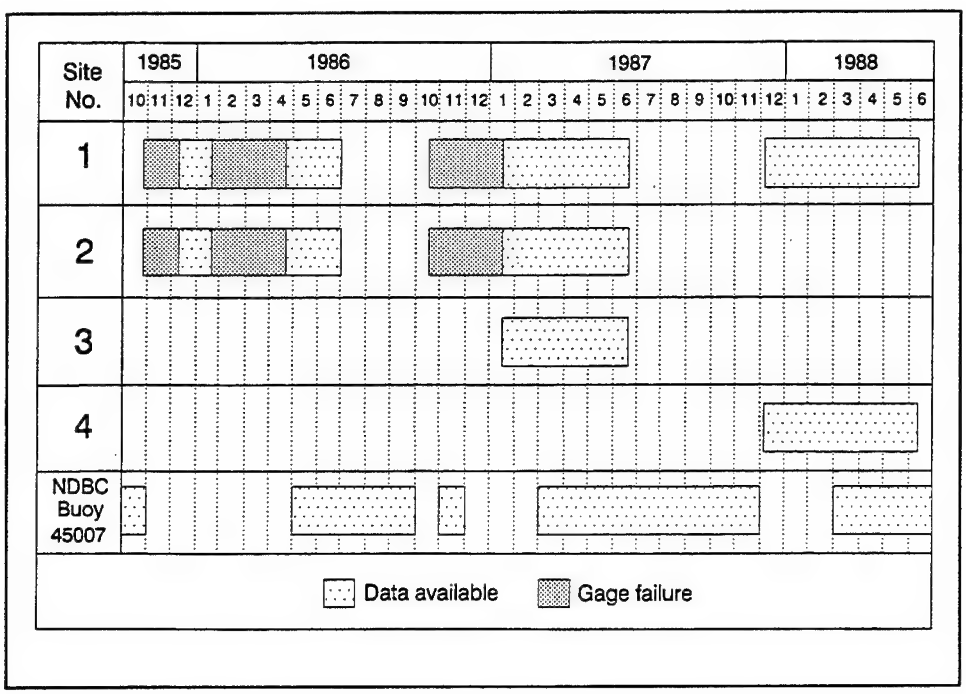

Figure 2-3. Wave gage deployments and data availability

Objectives

Objectives of the wave measurement effort were as follows:

a. Provide incident wave conditions for use in the structure stability investigation.

b. Establish and/or validate the wave climate for determining design conditions.

c. Establish transmission characteristics of the breakwater for comparison to model predictions.

d. Establish wave conditions at the grain dock.

e. Establish reflection characteristics of the breakwater.

Approach

Data required

Objective *a.* above required measurement of incident waves at the breakwater. Objective *c.* also required incident wave measurements, as well as simultaneous measurements behind the breakwater. Objective *d.* required measurements directly in front of the grain dock. Wave measurements directly in front of the breakwater were required for Objective *e.*

Objective *b* required a long-term wave record either at the structure or at a distant point from which a transformation can be developed. Hindcast wave conditions are available for 67 locations in Lake Michigan, the nearest being Station 62, approximately 10 n.m. (18.5 km) north of the harbor (see Figure 2-1). They are based on meteorological input to wave generation models, and are provided by the Wave Information Study (WIS), a work unit of the U.S. Army Engineer's Coastal Field Data Collection Program. At the initiation of the study, the hindcast covered the 20-year period from 1956-1975. During the course of the MCCP study, the hindcast for the Great Lakes was updated to 1987.

The NDBC buoy is removed each year during winter months (typically December through February) to avoid ice damage. A climatic summary was published for the measurement interval 1981-1988 (U.S. Department of Commerce 1990).

Two extremal analyses were conducted using the hindcast and measured data to provide estimates of probabilities of occurrence of incident wave heights. These analyses are contained in Chapters 3 and 5 of this volume.

Wave gages

CERC has found that bottom-mounted pressure transducers are reliable, rugged sensors that can measure relative water levels and wave conditions. Self-contained, single pressure gages (Sea Data model 635-11) were selected for the study because of availability and suitability for the environment (see Hemsley et al. (1991)). They were mounted on steel platforms on the lake bottom with the transducer 3 ft (0.9 m) above the bottom. The gages sampled the output of a quartz pressure transducer in 1-Hz bursts for 1,024 sec every 3 hr. A separate water level record is obtained by averaging 1-Hz samples continuously over a 450-sec interval. Data were recorded on magnetic cassette whose storage capacity permitted a deployment interval of about 3 months. When the instrument was retrieved, the tape was removed and returned to CERC for downloading to a VAX 11-750 computer using a Sea Data tape reader.

CERC has developed extensive procedures for data quality control and assurance which are performed on all measured data. Before performing analysis, the initial data quality is determined by inspecting the sensor's signals. Plots of pressure time series are viewed to determine if gages have malfunctioned. Records with problems such as plugged pressure ports, transducer drift, etc., are flagged for editing or removal.

Pressure data are also inspected for electronic noise, which usually appears as isolated large data "spikes" in the measured time series. Failure to eliminate spikes contaminates subsequent spectral analysis by loading higher frequency bands with unrealistic energy. Automated routines have been developed to check for the number and amplitude of spikes. Data values larger than what is physically possible are corrected using linear interpolation if spikes do not occur in sequence; otherwise the erroneous values are replaced with the record mean. If 10 percent of the total number of samples in a wave record are determined to be spikes, analysis of that record is discontinued.

Pressure time series are examined for stationarity prior to spectral analysis. Data are adjusted if linear trends, resulting from rising or falling lake levels, are identified. Records having higher-order trends are rejected.

Once data are spectrally analyzed (see below), results are examined to determine if they are realistic. Significant wave heights and peak periods are compared to available data from other sources for correlation. Spectra of suspect records are examined to determine if physical explanations are possible for differences from expected results.

Spectral Analysis

Overview of procedures

CERC has developed a standard procedure for estimating energy distribution in the frequency domain (the one-dimensional spectrum) using time series from pressure sensors (Earle, McGehee, and Tubman 1996). A sea surface power spectrum $S(f)$ may be obtained from a pressure spectrum by using linear wave theory. Details of theories for estimating a sea surface spectrum can be found elsewhere (e.g., Phillips (1977), Kinsman (1965)).

A Fast Fourier Transform (FFT) routine is used to compute the power spectra from the measured time series. The 1,024-sec time series is first divided into fifteen 50-percent overlapping segments. A 10-percent cosine bell window is applied to the beginning and end of each 1,024-sec time series prior to FFT analysis to reduce the undesirable effects of side lobes and spectral leakage in the transformation into frequency space. The energy spectra contain 64 frequency bands between the lower cutoff of 0.03 Hz and the upper cutoff of 0.24 Hz. The resulting frequency resolution is 0.00325 Hz.

The energy-based significant wave height Hm_o is estimated using the formula

$$Hm_o = 4\sqrt{\int S(f)df} \tag{2-1}$$

where $S(f)$ is band-pass filtered at the high frequency cutoff to reduce artificial effects of high frequency signals (electronic noise). A peak wave period T_p is defined as the inverse of the peak frequency f_p at which $S(f)$ has its maximum value.

Data accuracy

Like any instrument, a wave gage has a range of values over which it can be considered "accurate," within limits of uncertainty. While an individual sensor can be calibrated in a laboratory, the limits, threshold and maximum vary with site, gage placement and mounting, analysis techniques, and even wave conditions. The reduced wave parameters (height and period) are not measured directly, but are statistically derived estimates based on wave theories relating water surface elevation to pressure. The time series is transformed to the frequency domain, and a "peak" is selected from the spectrum to identify the period. If the spectrum is broad, or there are two or more modes of near equal energy, minute variations in signal strength can result in widely differing frequencies for "the" peak.

Another consideration is that the spatial variability of these reduced parameters over horizontal distances on the scale of meters may be large compared to the uncertainty in measurement. Meanwhile, the ability to position the gage horizontally, and thus specify the measurement point in space, is often no better than ± 32.8 ft (± 10 m). Thus, the question of accuracy of a field measurement is less meaningful than the applicability of the measurement over the spatial scale of interest. Additional uncertainty results from the degree of nonlinearity of the conditions at time of measurement, and thus is particular to a location and variable with time. There is no simple method of quantifying the combined uncertainty resulting from the linear assumptions, the sensor error, and the analysis process.

An illustration of expected uncertainty in reduced parameters for the data collected in this study can be obtained by examining the output from two wave gages placed on the same mount during one deployment. One gage is designated the primary, and the other the redundant gage, though the choice is arbitrary. Horizontal separation was 2 ft (0.6 m).

Figure 2-4 is a scatter plot of the significant wave height from the two gages, using a threshold of 0.5 m to eliminate low waves of no engineering significance. A 45-deg line provides a reasonable fit, with random variance evident on the order of 5-10 percent for the larger waves. Figure 2-5 is the ratio of the primary to the redundant gage wave heights as a function of peak frequency. With the exception of a few 4- to 5-sec period outliers (identifiable as barely over the threshold in Figure 2-4), there is no significant trend with period.

One possible source of error is seen in a sample of a demeaned pressure time series from both gages (Figure 2-6). A slight phase shift of 10 to 15 sec is apparent, likely resulting from imprecise notation of the gage's reset times. One of the assumptions in linear spectral analysis is stationarity of a random process over the measurement interval (~ 17 min), so a phase shift of this magnitude is considered inconsequential. Another potential source of variance is differences in hydrodynamic noise caused by the mounting arrangement, but this would be impractical to quantify.

A more significant difference is evident in the magnitude of the absolute pressure time series. Figure 2-7 shows the mean pressures from the water level record for both gages during their deployment. A constant difference of about 0.5 psi (0.003 MPa) between the mean pressures appears both in the air and underwater. This is well outside the manufacturer's stated accuracy, and indicates a defect in one or both pressure sensors. Since the pressure time series is demeaned prior to analysis, a constant bias will not result in an equivalent error in the analyzed products. However, the relationship between pressure and surface elevation assumed in the analysis is a function of depth. Figure 2-8 shows typical spectra from the two gages measured at the same time. Though there is some difference in the shape of the two curves, the resulting significant wave height and peak period agree well.

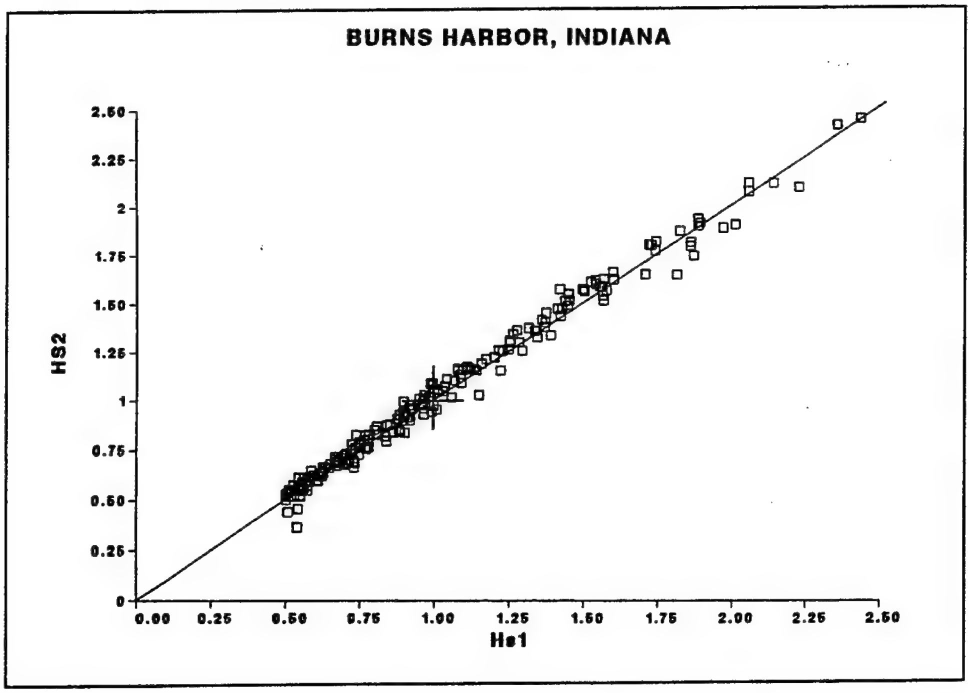

Figure 2-4. Significant wave height from primary (*Hs2*) gage versus redundant (*Hs1*) gage

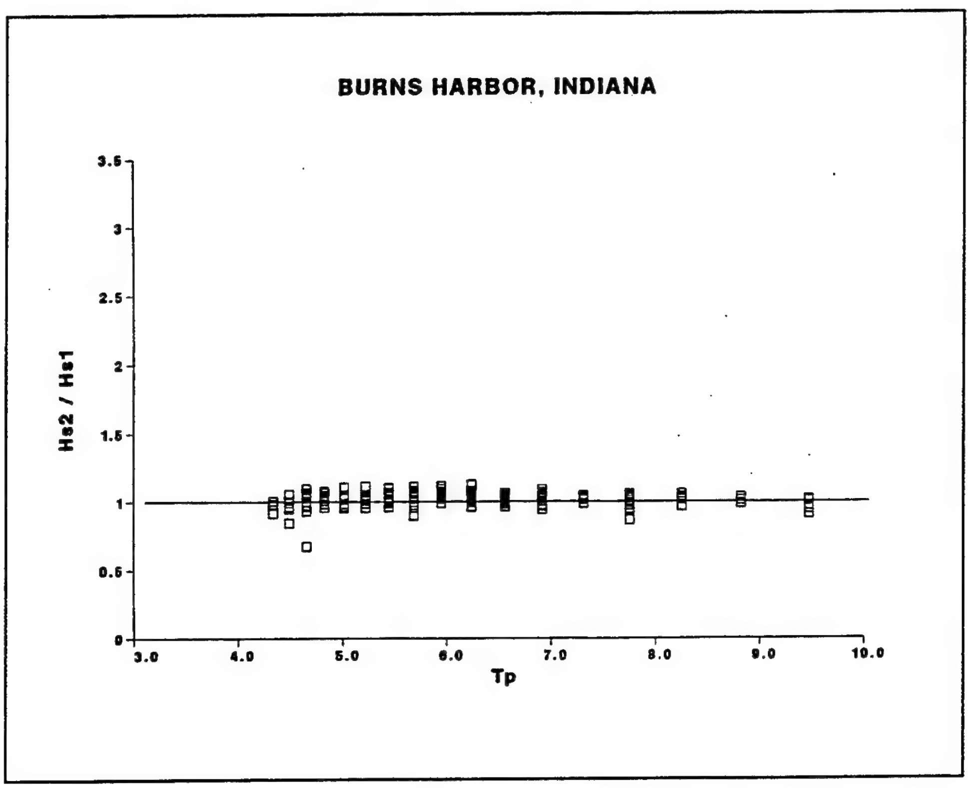

Figure 2-5. Ratio of primary to redundant wave height versus primary peak frequency

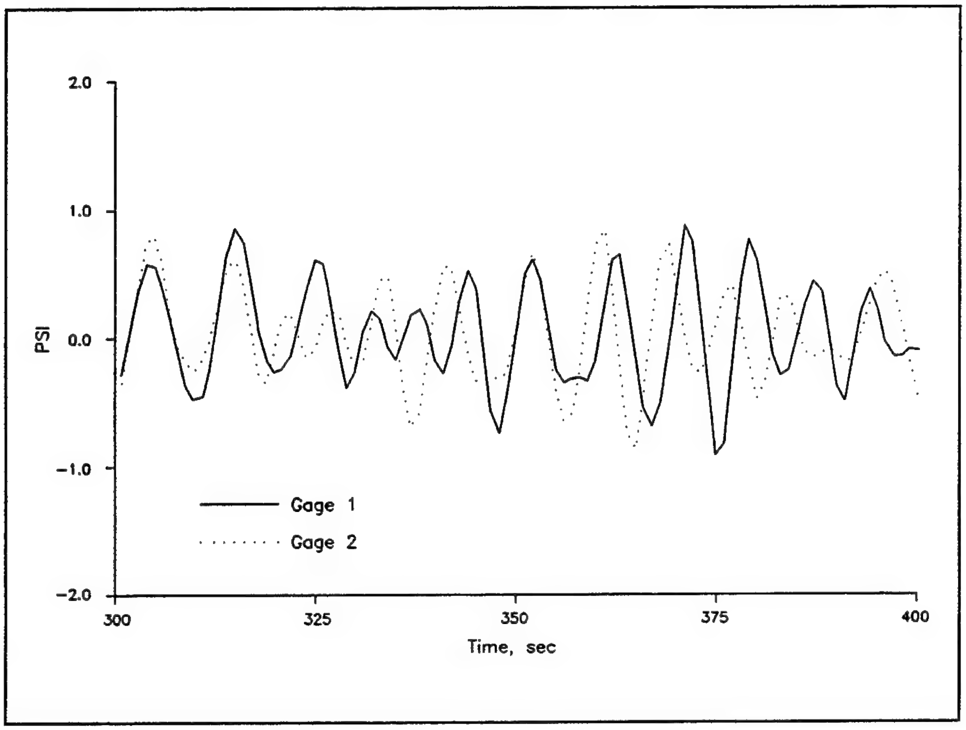

Figure 2-6. Demeaned pressure time series from primary and redundant gages

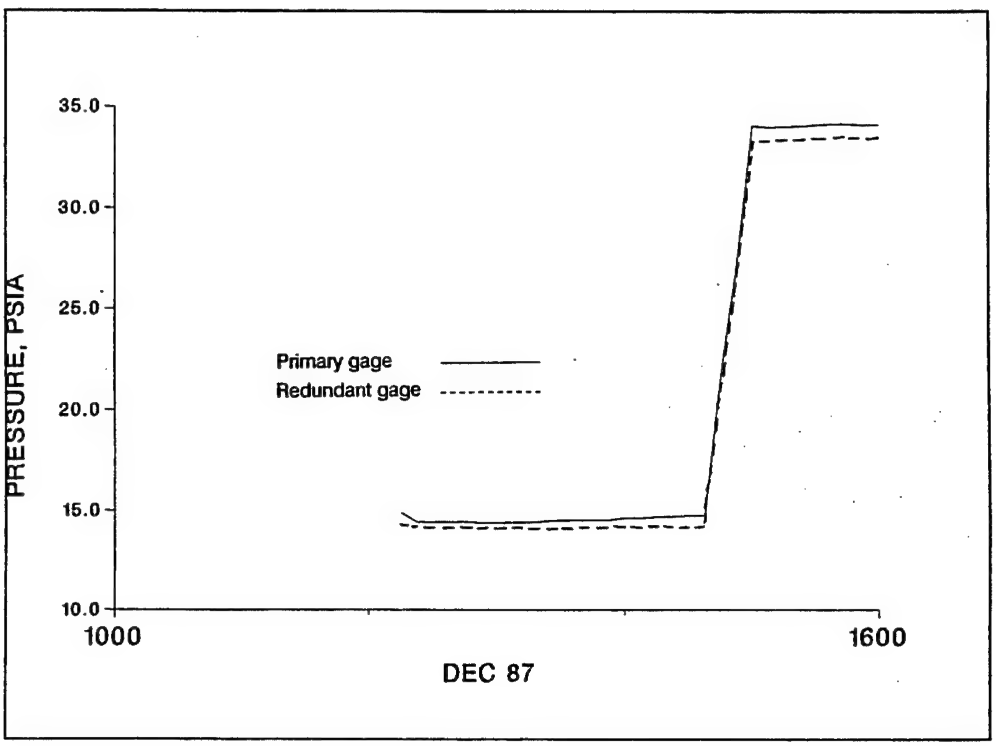

Figure 2-7. Pressure time series during deployment of primary and redundant gages

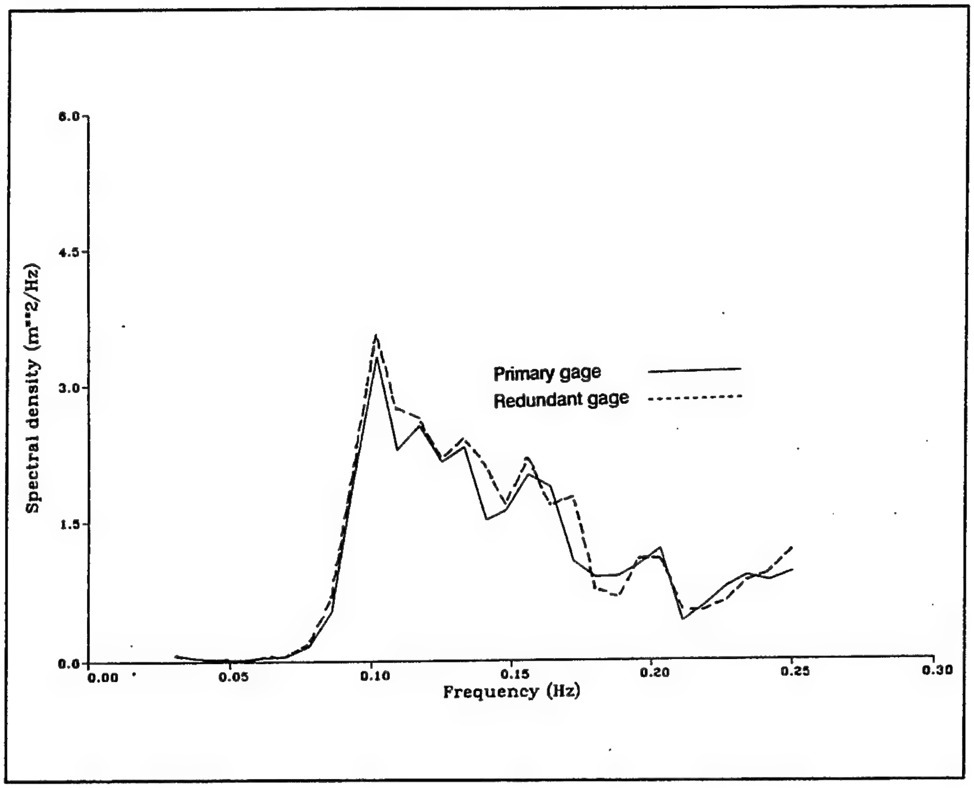

Figure 2-8. Energy density spectra from primary and redundant gages

A reasonable engineering estimate for the "validity" of the various measurements, given the size of the areas they are meant to represent, is as follows: for significant wave heights 10 percent and peak wave period ± 0.5 sec over the range of significant wave heights from 0.3 ft (0.1 m) to breaking. Caution and in-depth investigation into the complete time series and resultant spectra are advised before utilizing the reduced parameters beyond these estimated accuracies. Peak period, in particular, is subject to large differences resulting from minor variations in the energy distribution under certain conditions. These differences in period are not uncertainties in measurement, but rather artifacts of the definitions of the terms.

Wave/Structure Interaction

General

Wave reflection and transmission at rubble structures are complex phenomena that require simplifying assumptions to characterize. The reflection coefficient K_R and transmission coefficient K_T were originally defined for regular waves as the ratio of reflected and transmitted wave height, respectively, to incident wave height:

$$K_R = \frac{H_R}{H_I} \tag{2-2}$$

$$K_T = \frac{H_T}{H_I} \tag{2-3}$$

where

H_I = incident significant wave height

H_R = reflected significant wave height

H_T = transmitted wave height

Actual measurement of reflection and transmission has rarely been accomplished in the field, and most available information comes from laboratory experiments (*Shore Protection Manual*, 1984). Measurement of the transmitted waves, neglecting reflection effects behind the structure, is straightforward for both regular and irregular waves since the waves are translational. Measurement of the reflective wave height has always been problematical, even in the laboratory, because reflected waves are superimposed on the incident waves in front of the structure. When random waves meet a structure, all components of the spectrum do not reflect or transmit equally; the reflection and transmission characteristics are not constants, but functions of frequency. For any stationary irregular wave condition, the waves in front of the breakwater are not homogeneous. The incident and reflected components have non-random phase relationships, which violates the assumptions of spectral analysis.

A typical laboratory procedure for measuring K_R for regular waves involves calibrating the wave generator to the measured incident waves with the structure replaced by an absorptive material ("without structure" case), then measuring the incident plus reflected wave field at an antinode when the structure is present. A partial standing wave develops with an amplitude less than that caused by a 100-percent reflective vertical wall, i.e., twice the incident amplitude. The reflection coefficient is taken as the ratio of measured to perfect reflection. Effects of the waves reflected from the structure on the wave generator, and reflected from the side walls on the gage, are neglected.

The method of Goda and Suzuki (1976) can determine a reflection coefficient for each component of irregular incident waves. It involves the measurement of one-dimensional spectra at two locations, and calculation of a reflection coefficient K_j for each spectral component from the real and imaginary spectral coefficients. In practice, three or more gages are often used to provide multiple pairs, and the resulting values averaged (Seelig and Ahrens 1981). Once a frequency domain reflection function is defined, a

single parameter reflection coefficient can be calculated by taking the root mean square value of the spectral components.

Calhoun (1971) used essentially the same technique in field measurements at Monterey Harbor breakwater, a permeable, rubble-mound structure comparable in water depth to Burns Harbor. One pair of wave sensors in front of the structure was used to estimate incident and reflected spectral energy from the amplitudes and phase angles of the combined (incident plus reflected) signal at the two sensors. Only six relatively low-energy conditions were reported (waves less than 80 cm). The reflection coefficient function showed a non-linear dependence on frequency, but generally fell in the range of 0.5 to 0.7 for most of the energetic portion of the spectrum.

The assumption for both methods above is that the waves are approaching normal to the structure's face, since phase angles are a function of approach angle as well as the standing wave pattern. This assumption is valid in a flume, but may not be in the field. Though the larger waves at Burns Harbor were constrained morphologically to the northern quadrant, directional wave measurements would be required to accurately determine spectral reflection using this approach. The cost to obtain and analyze directional wave measurements at two or more locations in front of the breakwater to describe the spectral reflection characteristics was not justified.

The original monitoring plan specified placement of gages at Sites 1 and 2 (U.S. Army Engineer District, Chicago 1986), and the data obtained from these sites between December 1985 and June 1987 included several extreme events of importance to the structure stability and wave climate portions of the study. Measurements from Site 1, in front of the breakwater, contained the superimposed signal of the incident and the reflected energy, as described above. Determination of accurate reflection characteristics was not as high a priority as the transmission characteristics since reflection did not pose an operational problem at the harbor. It was important, however, to relate the transmission measurements and stability results obtained in 1985-1987 to an incident wave condition. Therefore, a procedure was needed that adjusted the observations at Site 1 to the actual incident conditions.

The spatial equivalent to a "without structure" laboratory measurement was provided at Burns Harbor by the presence of a natural beach just to the westward of the breakwater. Neglecting the reflected energy from the beach, a gage placed at the depth of the structure in front of the beach (Site 4) provides a reasonable incident wave condition at the structure.

If a true reflected spectrum, uncontaminated by incident wave energy, were available and the reflection process was assumed linear, a reflection transfer function could be determined from the ratio of the incident and reflected spectra. However, a single-parameter coefficient, not a transfer function, is required to meet the objectives stated above. The following section will use energy-based significant wave heights to determine single-value coefficients of reflection and transmission from the measured energy spectra.

Energy-based reflection

There is no simple way of directly relating the action of monochromatic and irregular waves on model rubble structures, but Rogan (1974) shows that equating irregular significant wave height with regular wave height is justified.

An equivalent reflection coefficient can be defined for irregular waves by using the energy relationship

$$H_{I+R}^2 = H_I^2 + H_R^2 \tag{2-4}$$

where H_{I+R} = significant wave height of combined incident and reflected wave field

In the following discussion, H_I will refer to data from Site 4, and H_{I+R} to data from Site 1.

Use of Equation 2-4 in this instance violates an assumption of spectral analysis: namely, that the phases of the n individual components, $a_k e^{i\omega_k t}$ (k = 1,2,...n) of the wave field, while random, are homogeneous over some finite space around the measurement point. That is, if the water surface elevation η is assumed to be of the form

$$\eta(t) = \sum_{k=1}^{n} a_k e^{i\omega_k t} \tag{2-5}$$

where

η = water surface elevation

t = time

a_k = amplitude of component k

ω_k = radian frequency of component k

Then the phases of the complex random variable are assumed uniformly distributed between 0 and 2π, and thus uncorrelated. The variance of η is then given by

$$\sigma_\eta^2 = \sum 1/2\, |a_k|^2 \tag{2-6}$$

where

σ^2_η = variance of η

i.e., the average energy, equals the sum of the average energies of the incident and reflected components. This is not valid if any pair of incident and reflected wave components are correlated, such as in the vicinity of the breakwater. In fact, Longuet-Higgins (1990) showed

$$\sigma_\eta^2 = \sum |a_k|^2 \tag{2-7}$$

for a wave field near a perfectly reflective wall, resulting in steeper waves and subsequently leading to breaking.

The purpose of the following analysis, then, is not to accurately define the actual reflection coefficient K_R of the breakwater, but to derive a relationship between the conditions measured at Site 1 and the true incident wave conditions. This site-specific K_R will be used to adjust the significant wave heights at Site 1 to a calculated significant wave height (hereafter referred to as Site 1A data) for use in deriving the actual transmission coefficient K_T. It is K_T that was predicted in the design phase, and is the parameter of interest to this study.

In the following analysis, two additional assumptions are made:

a. The actual reflection coefficient is less than 1; i.e.,

$$\sigma_\eta^2 \cong \sum 1/2|a_k|^2 \ , \ \neq \sum |a_k|^2 \tag{2-8}$$

b. Temporal average from Site 1 over a broad range of conditions (different distributions of frequency, direction, and spreading) will tend to smear the standing wave pattern and improve the estimate of K_R, just as the spatial averaging practiced by Seelig and Ahrens (1981) improved the estimate for a single wave condition.

The correlation between H_I and H_{I+R} is displayed in Figure 2-9 (a threshold of 0.5 was used to eliminate waves below engineering significance). The dotted line represents $H_I = H_{I+R}$, and seems to fit the observed data well when $H_I < 2$ m, suggesting reflection is negligible for small waves. As wave height increases, measurable effects of reflection can be seen. Most of the larger waves are associated with longer periods, so this verifies the expected trend of reflection with frequency.

To illustrate the gross effects of direction, wind measurements were obtained from WIS, and only those waves occurring when the winds were coming from northwest to northeast were retained (Figure 2-10). The effect is to remove some of the scatter of the smaller waves, but the trend is retained.

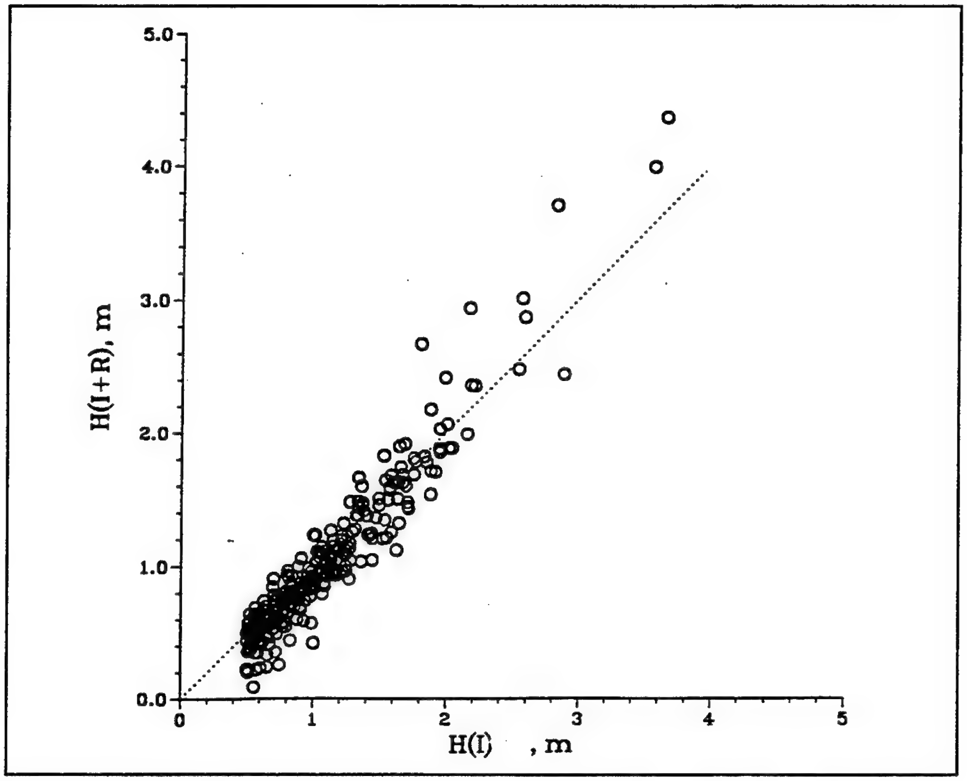

Figure 2-9. H_I (Site 4) versus H_{I+R} (Site 1) for all waves > 1.6 ft (0.5 m)

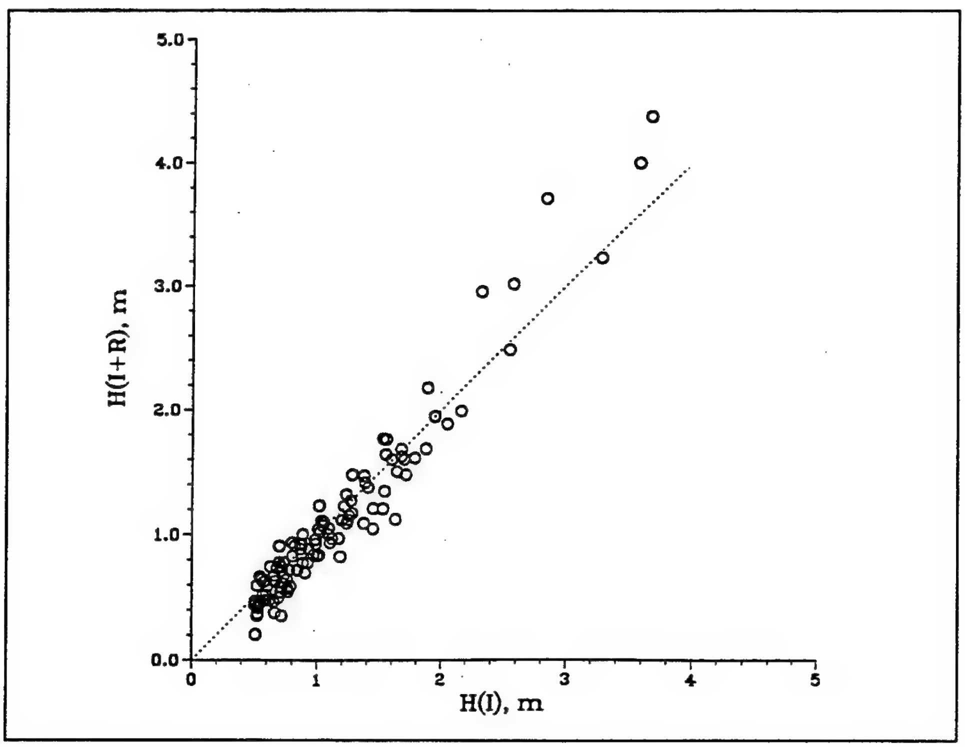

Figure 2-10. H_I (Site 4) versus H_{I+R} (Site 1) for waves > 1.6 ft (0.5 m) and measured when winds were from northwest to northeast

The calculation of K_R will be divided subjectively into two regimes by the 2-m incident wave height. Figure 2-11 shows the energy relationship (H^2) for the smaller waves. A linear regression fit appears almost as a 45-deg line, and can be expressed as

$$H_{I+R}^2 = 0.04 + 0.94\ H_I^2 \tag{2-9}$$

Figure 2-12 is the energy relation for incident waves > 6.6 ft (2 m). A second linear regression curve fits the data about as well as the low wave curve, and is expressed by

$$H_{I+R}^2 = -1.08 + 1.39\ H_I^2 \tag{2-10}$$

While a high order regression curve could be fit to the data, this is not necessarily a better description of the small number of data points over 6.6 ft (2 m). Thus, $K_R \cong 0$ for $H_I < 6.6$ ft (2 m). When $H_I > 6.6$ ft (2 m), an expression for K_R, using Equations 2-2, 2-4, and 2-10 is

$$K_R \cong 0.62 \sqrt{1 - \frac{2.77}{H_I^2}}\ ,\quad H_I > 2.0\ m \tag{2-11}$$

Equation 2-11 is plotted in Figure 2-13, with a dotted extension for $H_I <$ 6.6 ft (2 m). Typical values are comparable to the reflection measured at Monterey Harbor.

This relation can be used to produce a calculated incident wave record for the period December 1985 to June 1987, which is designated as Site 1A and is listed in Appendix 2A. It is this data set, together with the Site 4 data set, which is used for the stone stability and extremal wave analysis.

Energy-based transmission

Applying Equation 2-3, with H_T from Site 2, and H_I from Site 1A, provides the calculated K_T from the measured data. Figure 2-14 shows the transmission coefficient for $H_I > 1.6$ ft (0.5 m). Up to $H_I = 6.6$ ft (2 m), the mean value of K_T remains close to 0.16, but increases gradually with H_I to a maximum of near 0.34. Figure 2-15 shows the same data limited to those times when the wind was blowing from the northerly quadrant. The trend is virtually identical. A second-order regression curve, which fits the entire range of wave heights reasonably, is defined by

$$K_T = 0.192 - 0.053\ H_I + 0.018\ H_I^2 \tag{2-12}$$

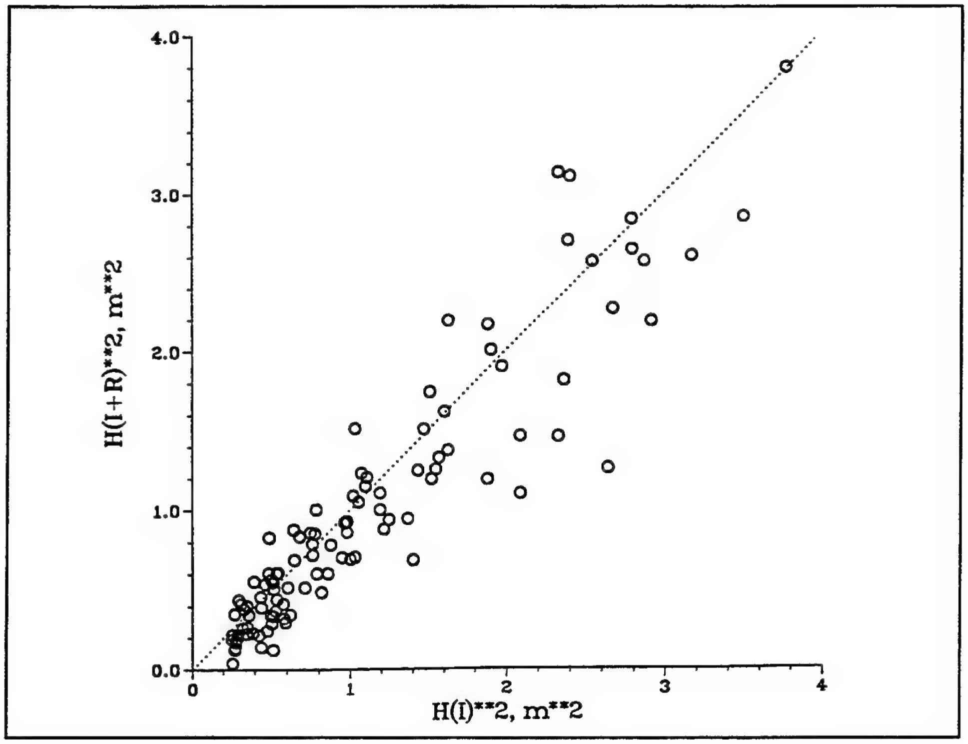

Figure 2-11. $(H_I)^2$ versus $(H_{I+R})^2$ for waves between 1.6 and 6.6 ft (0.5-2 m); measured when winds were from northwest to northeast

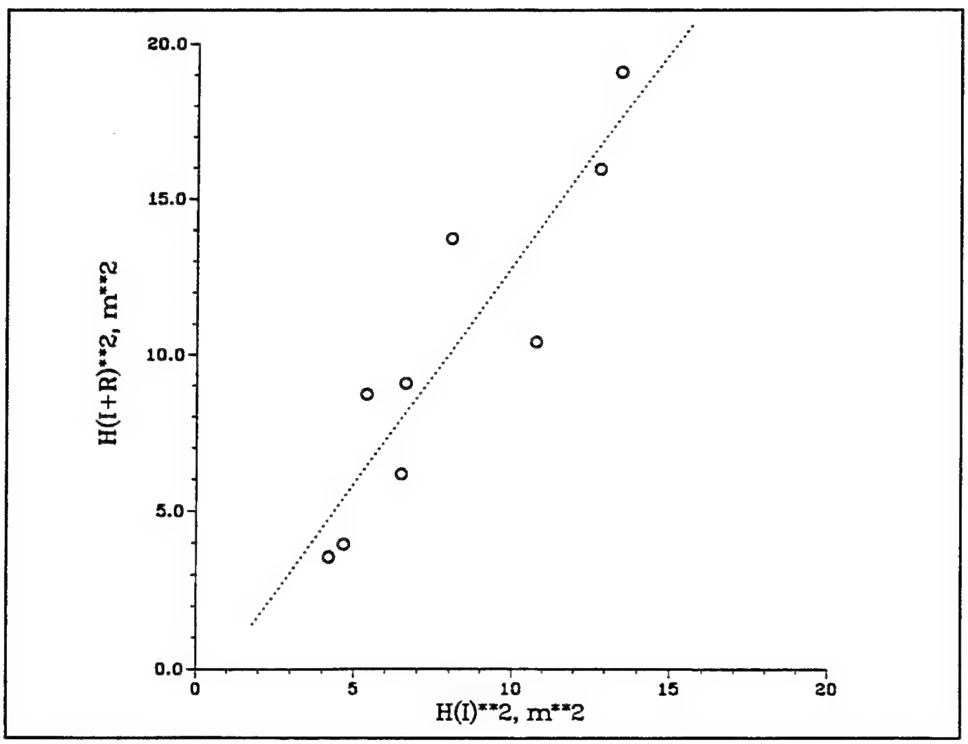

Figure 2-12. $(H_I)^2$ versus $(H_{I+R})^2$ for waves > 6.6 ft (2 m) and measured when winds were from northwest to northeast

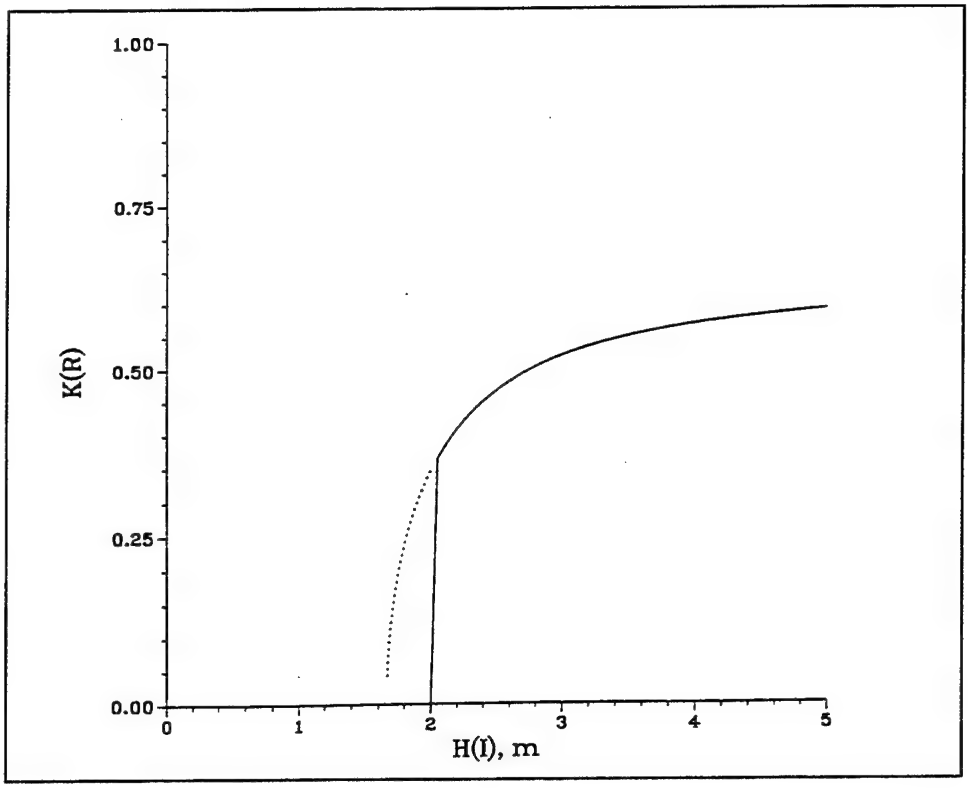

Figure 2-13. K_R versus H_I for waves > 6.6 ft (2 m) and measured when winds were from northwest to northeast

It is possible to show general trends of the dependence on K_T with frequency by illustrating the effect of peak period T on the transmission (Figure 2-16). Expected growth of K_T with wave period is verified, but there is considerable scatter since there can be a wide range of significant wave heights associated with any one period. Height and period can be combined in the dimensionless wave steepness parameter H/gT^2 where g is the gravitational acceleration. Figure 2-17 shows a general trend for decreasing transmission for increasing steepness. This would be expected as the steeper waves would lose more energy to dissipation. However, the highest values (identifiable as due to the highest waves) cluster around an intermediate steepness, showing that steepness alone is not the controlling criterion.

Another way of combining height and period is provided by the wave power $H_I^2\ T$. Figure 2-18 plots K_T with wave power, with larger, longer waves increasing to the right. The result is a more dichotomous grouping of the data into low-power and high-power conditions, with a sparsity of the latter. Though it would require more data to be conclusive, the reduction of scatter in K_T makes wave power an intriguing candidate for classification of transmission characteristics.

Finally, to illustrate the variance of the available data, a plot of the probability of exceedance of K_T is shown in Figure 2-19, for all measurements, regardless of wind direction. Note that 99 percent of

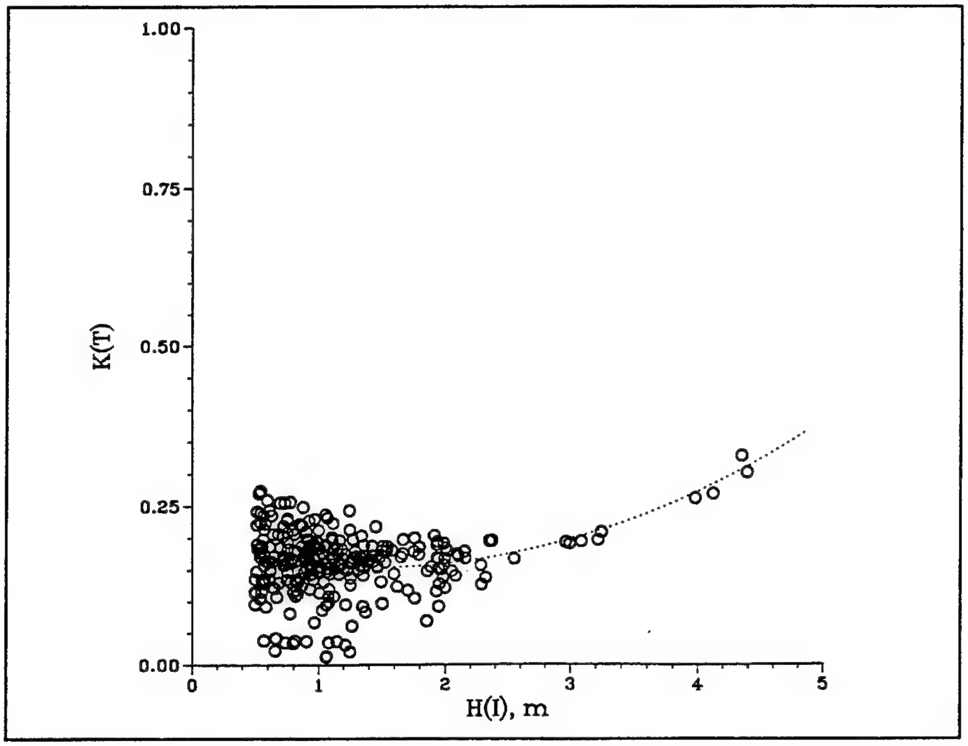

Figure 2-14. K_T versus H_I (Site 1A) for all measured waves > 1.6 ft (0.5 m)

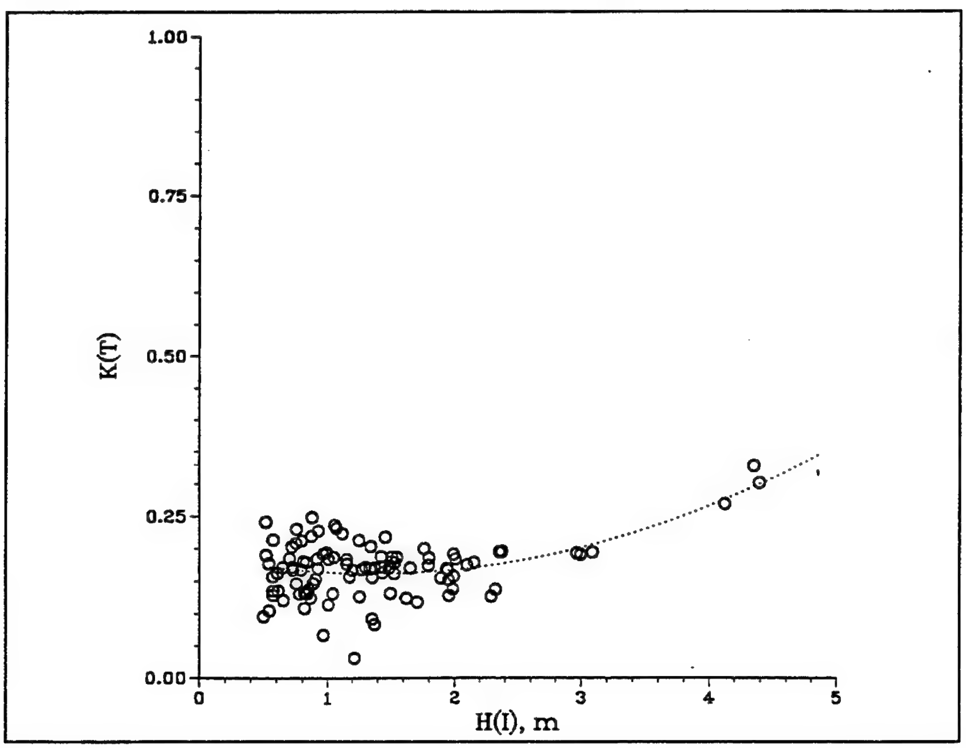

Figure 2-15. K_T versus H_I (Site 1A) for waves > 1.6 ft (0.5 m) and measured when winds were from the northwest to northeast

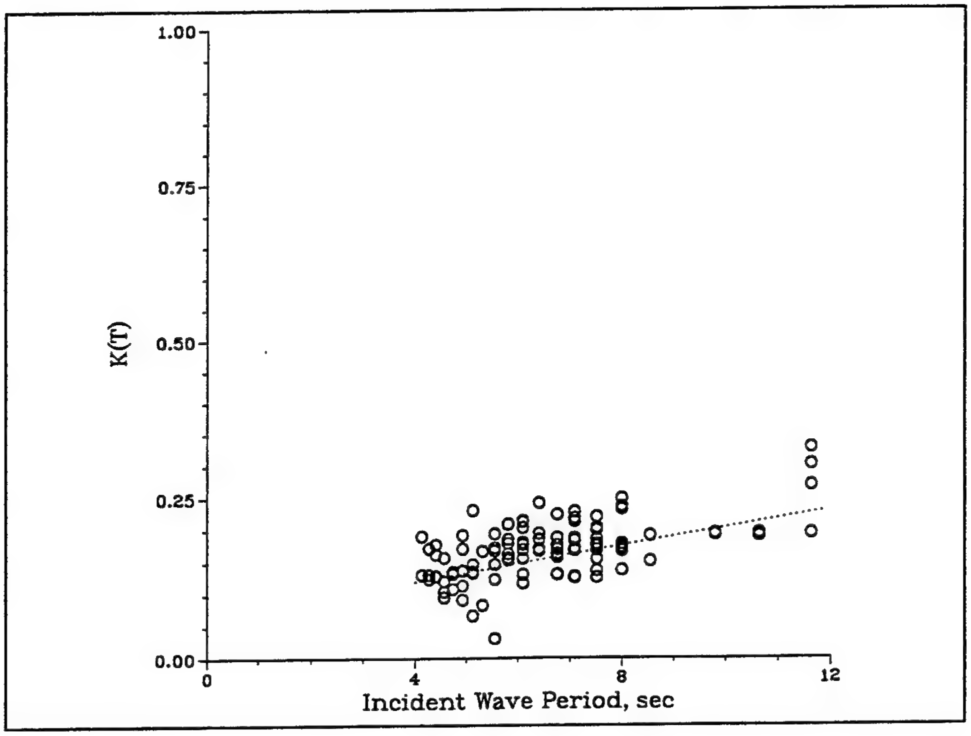

Figure 2-16. K_T versus T_p for waves measured when winds were from the northwest to northeast

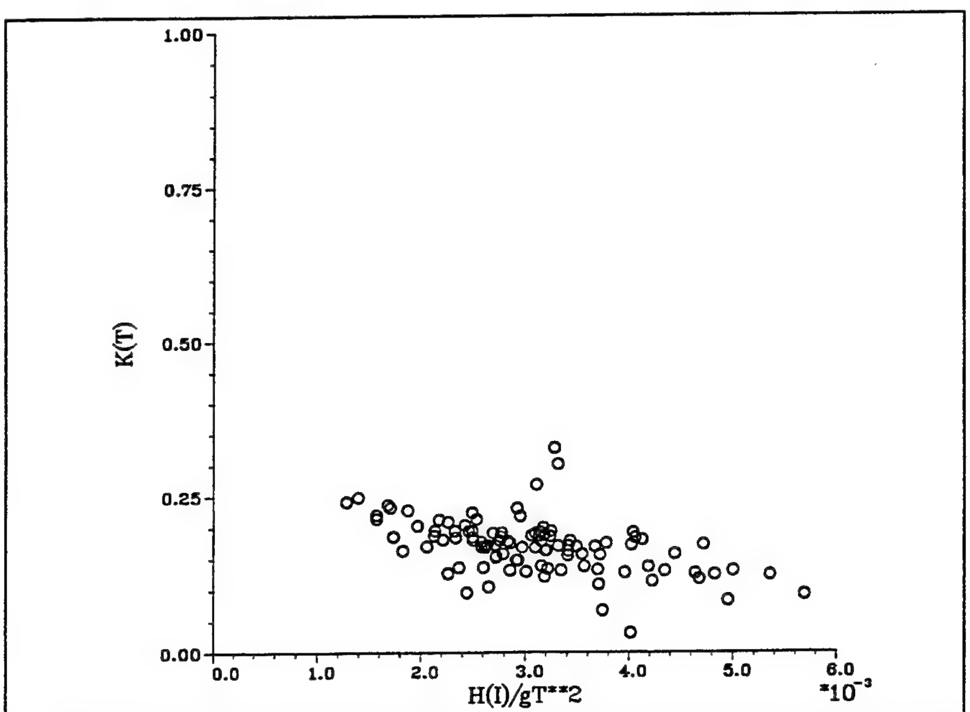

Figure 2-17. K_T versus incident wave steepness (H/gT_p^2) for waves measured when winds were from the northwest to northeast

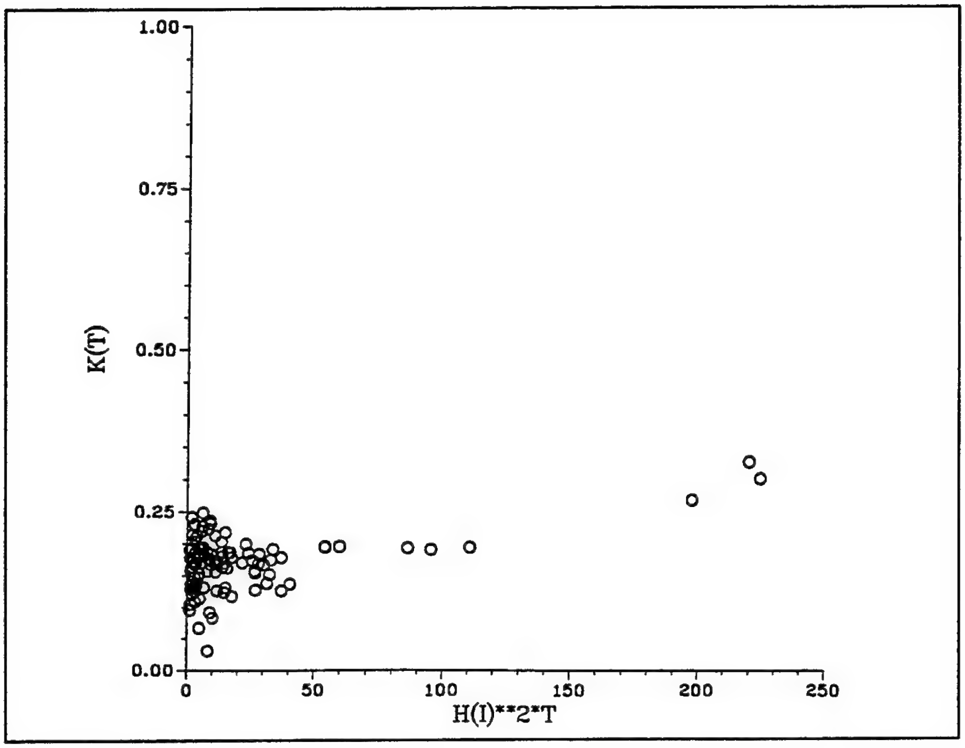

Figure 2-18. K_T versus wave power $(H_i^2 T_p)$ for waves measured when winds were from the northwest to northeast

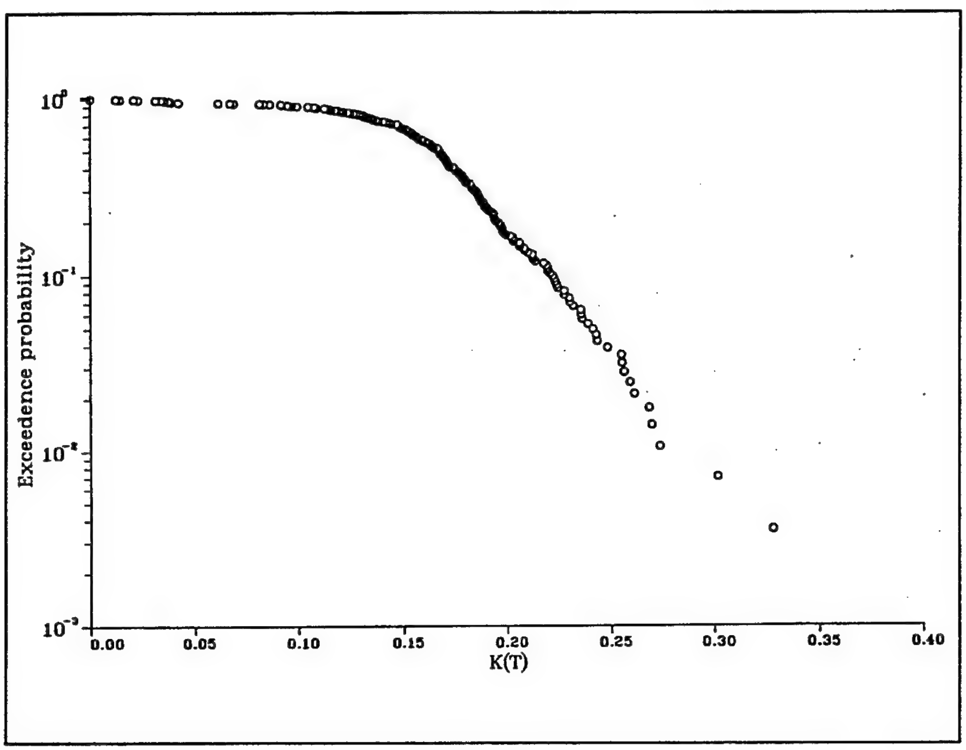

Figure 2-19. Exceedance probability of K_T for all measured waves

the 283 observations show the transmission coefficient less than 0.27, and 90 percent less than 0.22. The general trend and dependence of K_T on wave height or wave power have been illustrated for the observed waves less than 14.8 ft (4.5 m) in height. Extrapolation beyond the range of present observations should be done with caution.

Energy dissipation. The remaining energy not reflected or transmitted can be assumed dissipated and represented by a dissipation coefficient K_D:

$$K_D = \sqrt{1-(K_R^{\,2}+K_T^{\,2})} \tag{2-13}$$

The ratio of dissipated to incident energy is K_D^2. For example, when H_I = 4.0 m, K_D = 0.78. The nonlinearity of Equation 2-13 allows the transmission or reflection to be very sensitive to dissipation. For example, if K_D was increased just 5 percent from 0.78 to 0.82, while the reflection remained constant, the transmission coefficient would drop in half.

Waves at grain dock

The grain dock is faced with a vertical bulkhead, which should provide a 100-percent reflecting surface for wave energy. The wave height measured by a gage in a standing wave pattern depends on its position x from the reflecting wall, and is harmonic on $2\pi x/L$, where L is the wavelength. Figure 2-20 is a plot of the measured and predicted ratio of the standing wave height to the incident height as a function of incident wave period. When $x = 15$, the predicted curve goes to 0 at the node, $x = L/4$, corresponding to $T \cong 6$ *sec.*

The measured data is observed to agree well with the predicted curve, verifying that the bulkhead is a "perfect" reflector. Wave heights at the face of the dock are about twice the amplitude of those at other locations in the harbor not experiencing significant reflection.

Vessels that are large relative to the incident wave length, such as an ore carrier, respond to wave forces in complex, nonlinear fashion. A vessel that is small relative to the wavelength, such as a barge, will respond to waves more like a surface-following body. A barge moored at the grain dock will experience excursions on the order of the wave height, and slopes on the order of the wave slope, of the incident wave. For a 1-m, 11-sec incident wave in the harbor, the wave height at the dock will be on the order of 2 m, and the slope of the water surface near 2 deg. This slope will induce a gravitational component to the mooring force which will be about 4 percent of the vessel displacement, or on the order of 40,000 lb (18,144 kg) for a loaded barge. If there is sufficient slack in the mooring lines, inertial forces on the same order or greater will also be induced. Combined reaction loads could exceed the breaking strength of several 1-in. nylon mooring lines.

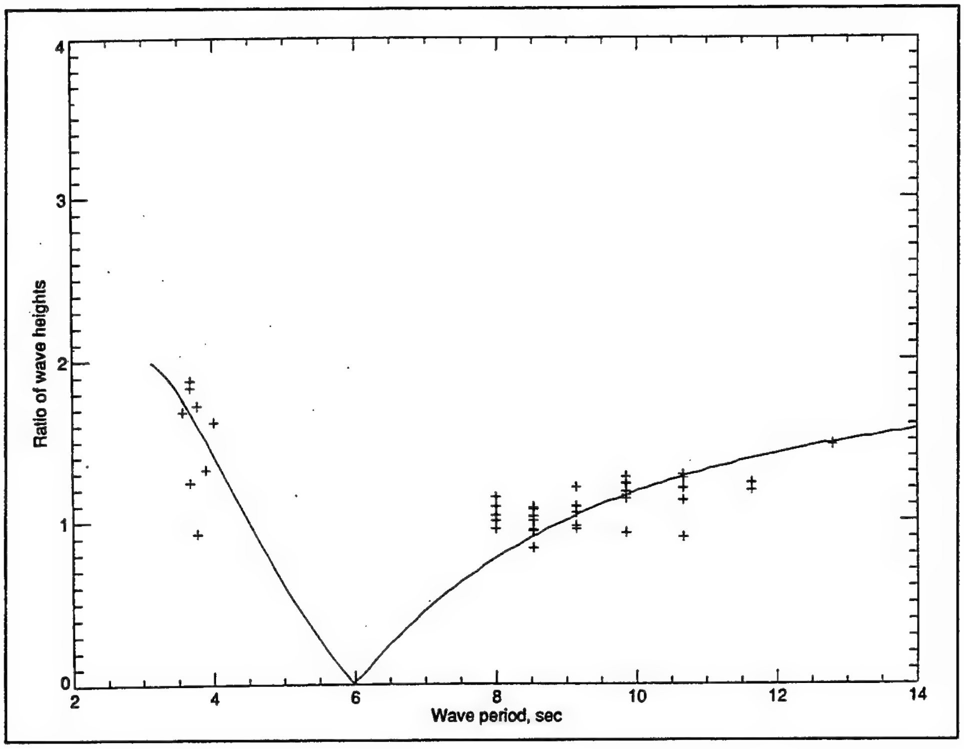

Figure 2-20. Measured (+) and predicted (----) reflection coefficient versus wave period

Model Comparison

The breakwater cross-section design was based on the results of a 2-D, 1:35 scale physical model tested at WES (Jackson 1967). The tests were conducted at the WES 119-ft-long by 5-ft-wide by 4-ft-deep (36.3- by 1.5- by 1.2-m) flume using a regular, plunger-type wave generator. A concrete floor extended 72 ft (22 m) (model dimensions) from the toe of the breakwater at a slope of 1:100. The study predicted wave transmission for eight plans. The report's conclusions recommended Plan 8 as the optimum, and the breakwater was constructed using this cross section.

Incident wave heights in the flume were measured at one site in front of the structure, and transmitted wave heights at two locations behind the structure - one at 1/2 L and one at L, measured from the structure center line, where L was derived from the wave period using shallow water linear wave theory. Two water levels, 0 and +4 ft (1.2 m) low water datum (LWD), and three incident wave periods, 7, 9, and 11 sec, were tested for various plans, but only 11-sec waves were tested for Plan 8. Wave heights tested for Plan 8 ranged from 9 to 18 ft (2.7 to 5.5 m), prototype scale.

The study did not specify the procedures used to measure the incident waves, particularly procedures to ensure the incident measurements did not include the reflected energy. Two options have been used in other model studies using a single incident wave gage to address this problem. The first is pre-calibration of the wave paddle stroke using an absorbing material in place of the structure. This could introduce error when the structure is in place due to increased loading on the paddle from the reflected waves, and, for the larger wave heights, wave-wave interactions. The second method can be used with the structure in place by using only the first few waves in the train, measured before the reflected waves can return to the gage. This produces a small sample of wave heights, the first of which is typically not representative of the rest. Without additional details on the technique used, these potential sources of error cannot be estimated.

Figure 2-21 is a plot comparing the prototype and model transmission characteristics. A cutoff below 0.5 m was used to eliminate waves of no engineering significance. Water levels during the time when the data were collected were obtained from the Calumet Harbor Lake Level Gage (#7044) operated by CENCC. The average annual lake level was 4.33 ft (1.3 m) (LWD) in 1986 and 3.31 ft (1.0 m) in 1987, so the model results from the +4-ft (1.2-m) test are appropriate for comparison. Actual transmitted measurements were made at a distance of about 75 m from the center of the breakwater. This location falls halfway between *L/2* and *L* for the longer waves of interest (*L* on the order of 328 ft (100 m)). It is not certain that the increasing trend with distance behind the structure exists in the prototype, so both the *L* and *L/2* model data sets will be retained.

Prototype data ranged from 4.1 to 11.6 sec, but only those prototype waves with periods greater than 10 sec, comparable to the 11-sec model waves, are included in Figure 2-21. The transmittance of the prototype structure as a function of incident wave height is best represented by plotting against data from Site 1A (corrected for reflected energy). However, since it is not known how or even if the model study corrected the incident measurements for reflected energy, the corresponding values for Site 1 (uncorrected) will also be plotted.

Only seven prototype incident wave data points meet the criteria of having periods greater than 10 sec, and these are compared to ten modeled incident wave heights. The general trend of both data sets is increasing transmission with increasing wave height, though the prototype data from Site 1A reveals higher transmission, with a maximum of 0.35 for the 14.3-ft (4.35-m) incident wave; the model reaches 0.32 for the 18-ft (5.5-m) wave. The data from Site 1 more closely follow the model results.

Transmission is more significantly underpredicted for waves below 10 ft (3 m). Long-period waves below 10 ft (3 m) are transmitted through the structure without overtopping, and are strongly influenced by the structure's porosity. Results of later research showed that the core material should be

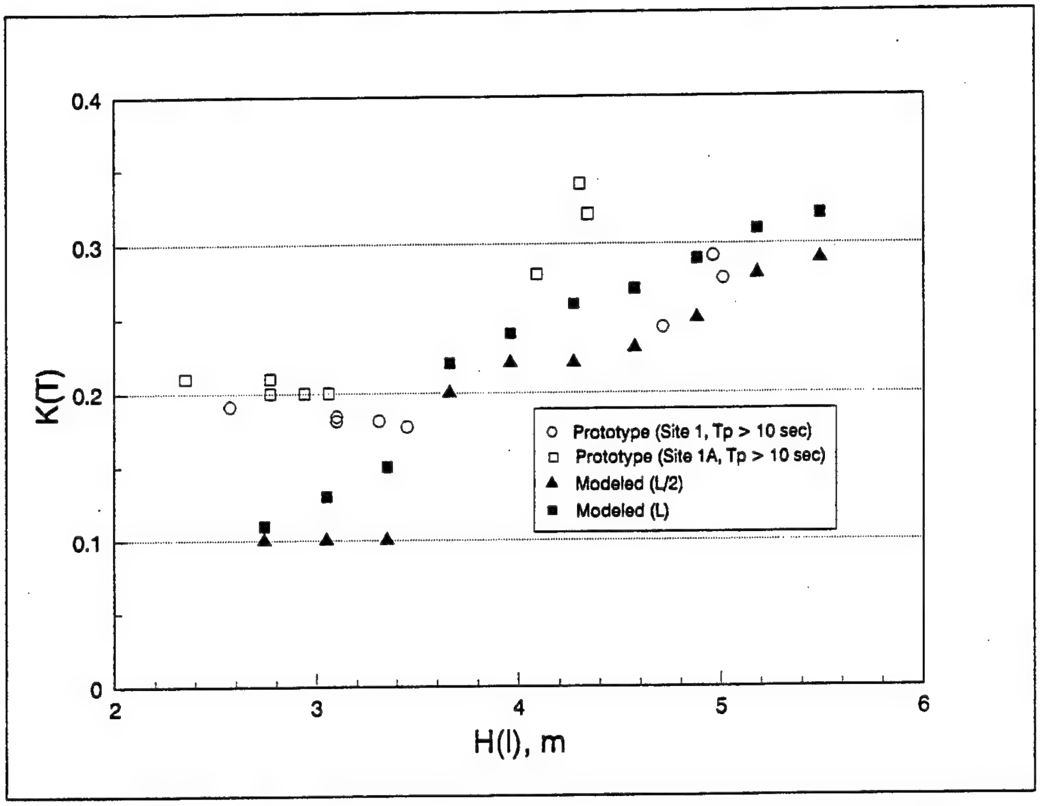

Figure 2-21. Prototype and model incident versus transmitted wave heights

oversized relative to the linear scaling relationship to compensate for viscosity effects (Keulegan 1973). The core material in the 1966 study was sized linearly, like the cover layers, and Figure 2-21 illustrates the effect of the increased viscous drag, relative to the prototype, at these scales.

Another factor that would tend to increase the measured energy at Site 2, and thus the prototype transmission coefficient, is the effect of energy coming through the entrance, in spite of the attempt to minimize this influence by its position. Finally, the lake level during the more extreme events exceeded the 1.2-m (4-ft) LWD used in the model study. The measured data for waves over 10 ft (3 m) are from a storm that occurred on February 8-9, 1987, and March 9, 1987. Lake levels for the February storm, as measured at the Calumet gage, exceeded 6 ft (1.8 m) LWD. This increased water level undoubtedly affected the transmission. Lake levels at Burns Harbor during the March event, as measured at Site 1, were very near the 4.0-ft (1.2-m) LWD used in the model study.

Evaluation of the model's performance presupposes that the model cross section duplicates the prototype; i.e., that the actual structure was constructed as designed. The stability analysis has shown that significant amounts of armor have been added to the structure without a concomitant increase in structure elevation or volume. Therefore, the existing structure must contain a higher percentage of armor, and the less porous layers must be correspondingly lower in the cross section than the design structure. Whether this increased porosity is sufficient to account for the increased transmissivity

cannot be determined with the existing data, but it is certainly a contributing factor.

The model data show an abrupt discontinuity around 11.5 ft (3.5 m). It is near this point that the model study indicated overtopping occurred. It seems likely the additional energy coming over the model structure caused the increase in total transmittance. There is not as obvious a jump in the prototype data, though it could be argued that an increase in the rate of transmittance occurs between 3 and 4 m (depending on whether data set 1 or 1A is used). It is likely that this corresponds to the onset of significant overtopping in the prototype as well. The model transmittance compares better with the prototype in the combined transmission/overtopping regime.

Direct comparison of the model and prototype is hampered by the following factors:

a. Model data.

(1) Regular waves.

(2) Uncertainty in incident wave height (effect of reflection on measured height is unknown).

(3) Scale effects (core sizing).

b. Prototype data.

(1) Irregular waves.

(2) Uncertainty in incident wave height (on the order of 1 m).

(3) Peak periods different from model wave periods.

(4) Transmitted gage position different from model.

Within the (quantifiable) uncertainty of the prototype incident wave heights, the model agrees with the prototype measurements for waves over 11.5 ft (3.5 m).

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Appendix 2A Wave Gage Technical Specifications and Measured Wave Data Plots

Sea Data, Inc. Mdl. 635-11 Wave and Tide Recorder

Pressure Sensor: Paroscientific, Inc., "Digi-Quartz"

	100 psia*	
	feet	meters
Standard Ranges:	190	58
Maximum Depth:	235	70
Resolution - Waves:	0.0035	0.10 cm
Tides:	0.0040	0.12 cm

Accuracy

(more than 80 ft)	0.03
(less than 80 ft)	0.05
vs temp @ 30 ft	0.004 ft/°C (max)

Frequency Response: DC to 1.0 Hz (Nyquist limit for 0.5-sec sampling)

Stability:

vs time:	0.0002 percent FS/month at (almost constant) ocean depths
vs temp:	zero 0.0007 percent FS/°C span 0.005 percent FS/°C (at 2/3 FS, 0.004 percent/°C)

Timebase:	4.194304 MHz special quartz crystal
Stability:	0.1 ppm/°C, 1 ppm/year; unmeasurable (0.001 percent) pressure data error at ocean depths

Physical Specifications:

Size:

Case:	7-in. diam. by 24 in. long
Mounts:	two 0.5-in. bolt holes on 13-in. centers, 1.0-in. clearance

Weight: 41 lb in air, with battery; 12.5 lb in water

Pressure Case:

Material:	6061-T6 aluminum
Hardware:	316 stainless and Delrin insulators
Finish:	Hard-coat anodize with electrostatic epoxy overcoat
Depth:	1,100-m operating depth

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* psia = pounds per square inch, absolute.

Description of Wave Data Parameters

1. Appendix 2A consists of time-series plots of wave height, period, and depth.

2. The parameters included are defined as follows:

   a. Wave height, H_{mo}: Energy-based significant wave height; equivalent to statistically-based significant wave height for deepwater waves. See the *Shore Protection Manual* (SPM), Vol. II, p. B-5.

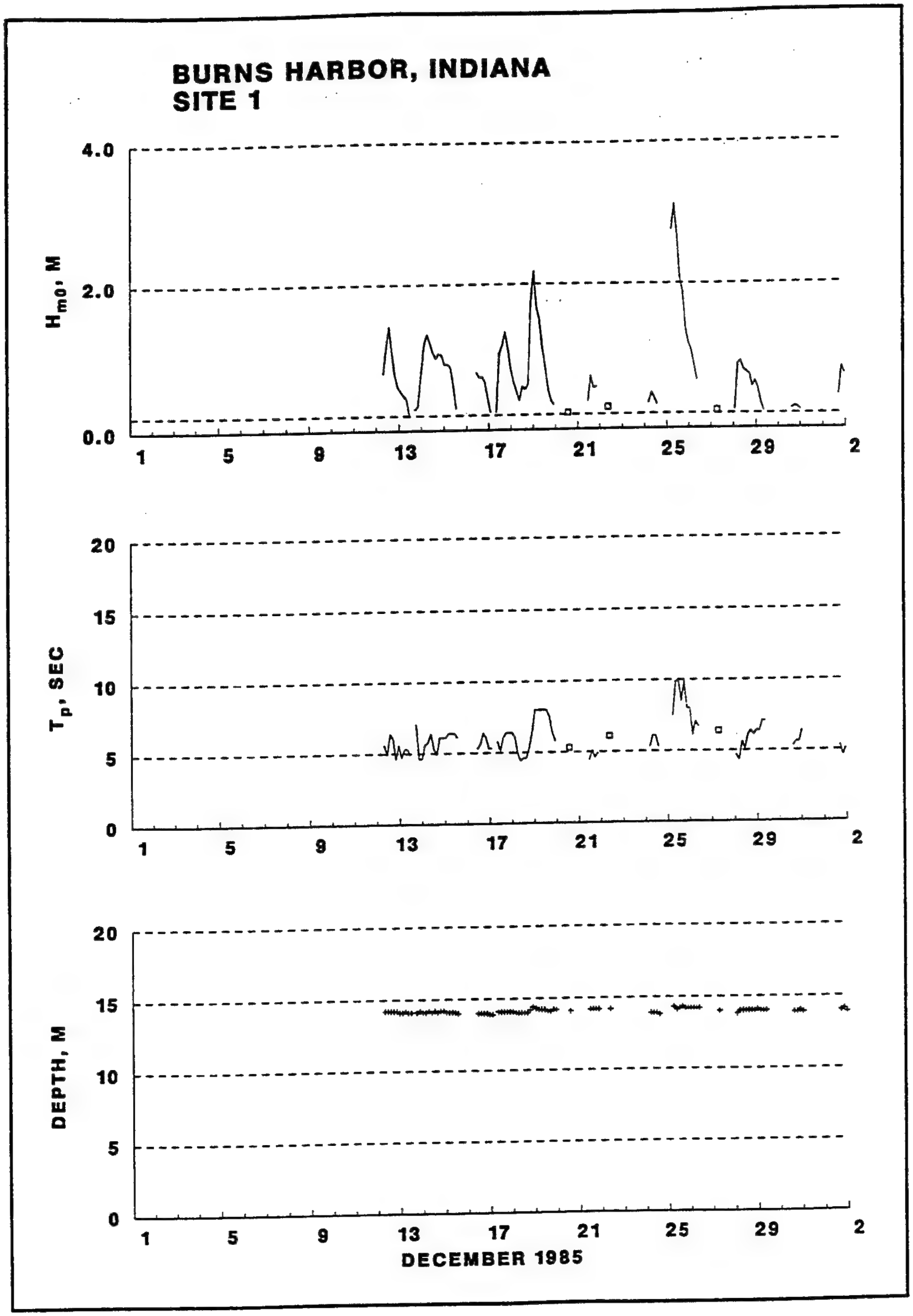
   b. Wave period, T_P: Peak spectral period; inverse of the dominant (highest wave energy) frequency of a wave energy spectrum (SPM, Vol. II, p. B-14).

   c. Water depth, Depth: Surface-to-seabottom depth at the gage location; average value of the sample burst.

3. Missing Data and Quality Control

   a. Missing data and data that failed to pass quality control measures do not appear in the time series plots.

   b. Just as it is difficult to determine wind direction for very low wind speeds, it is difficult to calculate wave parameters during times of low wave energy. When wave heights are low, the computed parameters may not be meaningful; therefore, when the calculated H_{mo} is less than 0.2 m, the data are omitted from the time series plots.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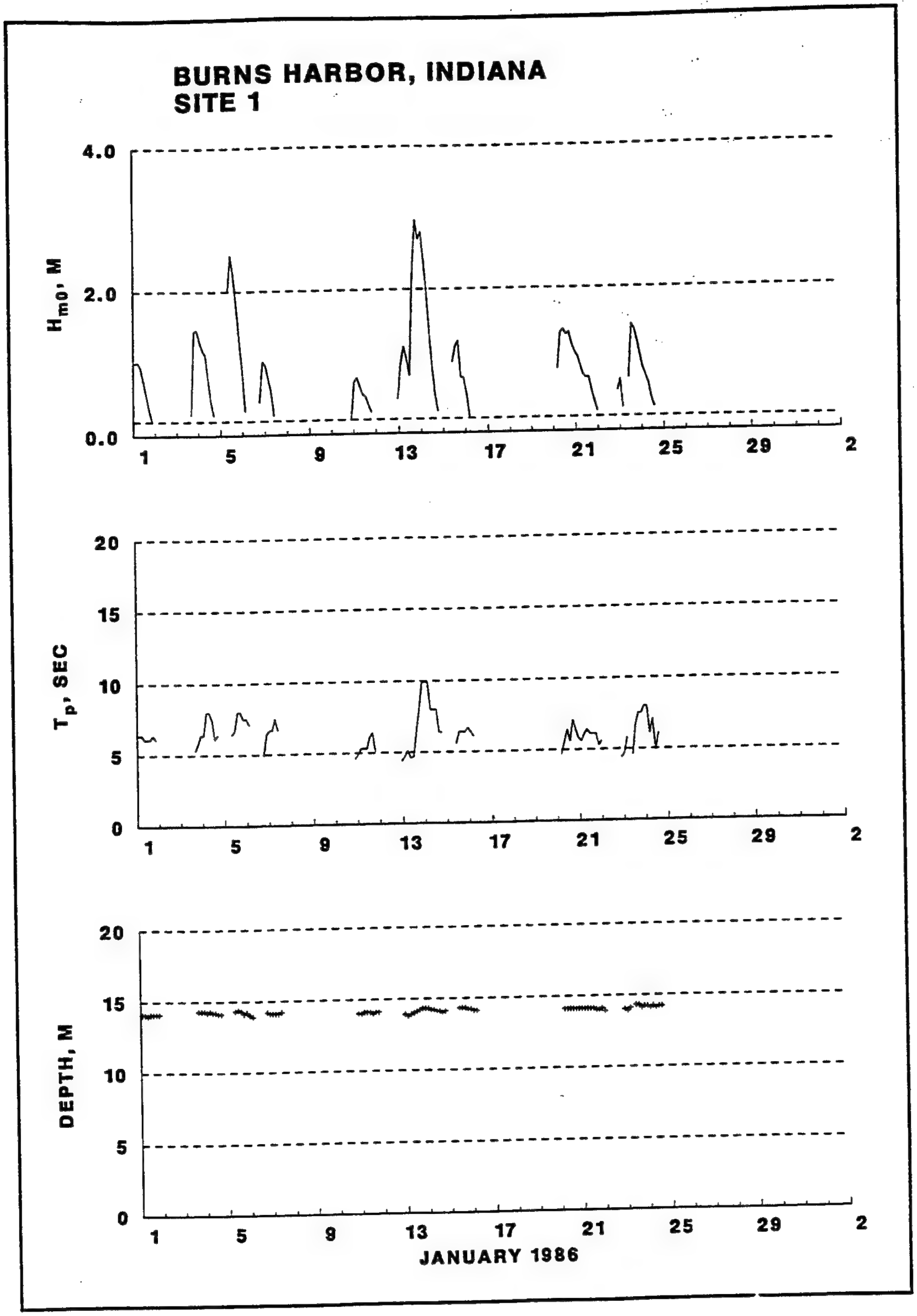
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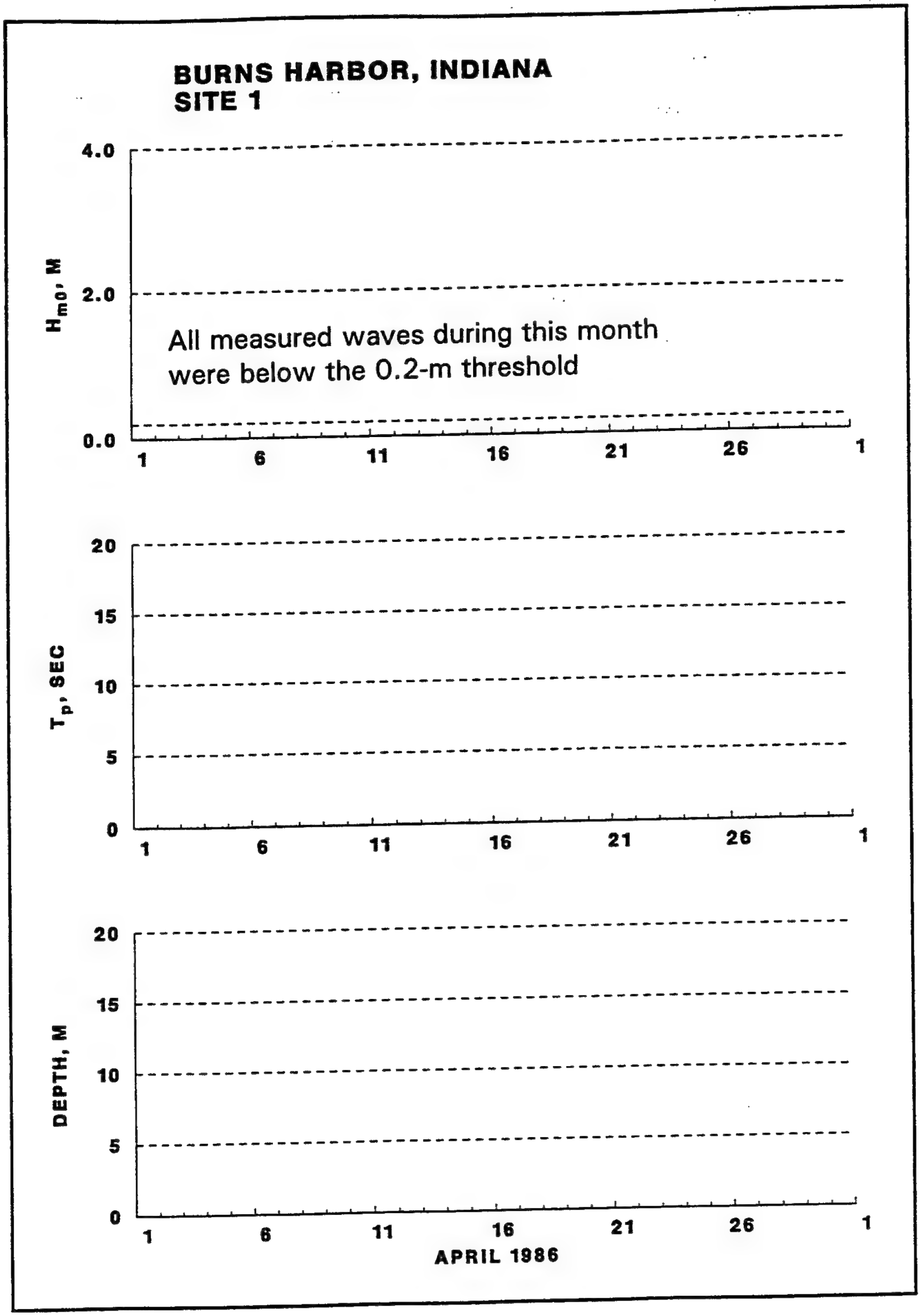
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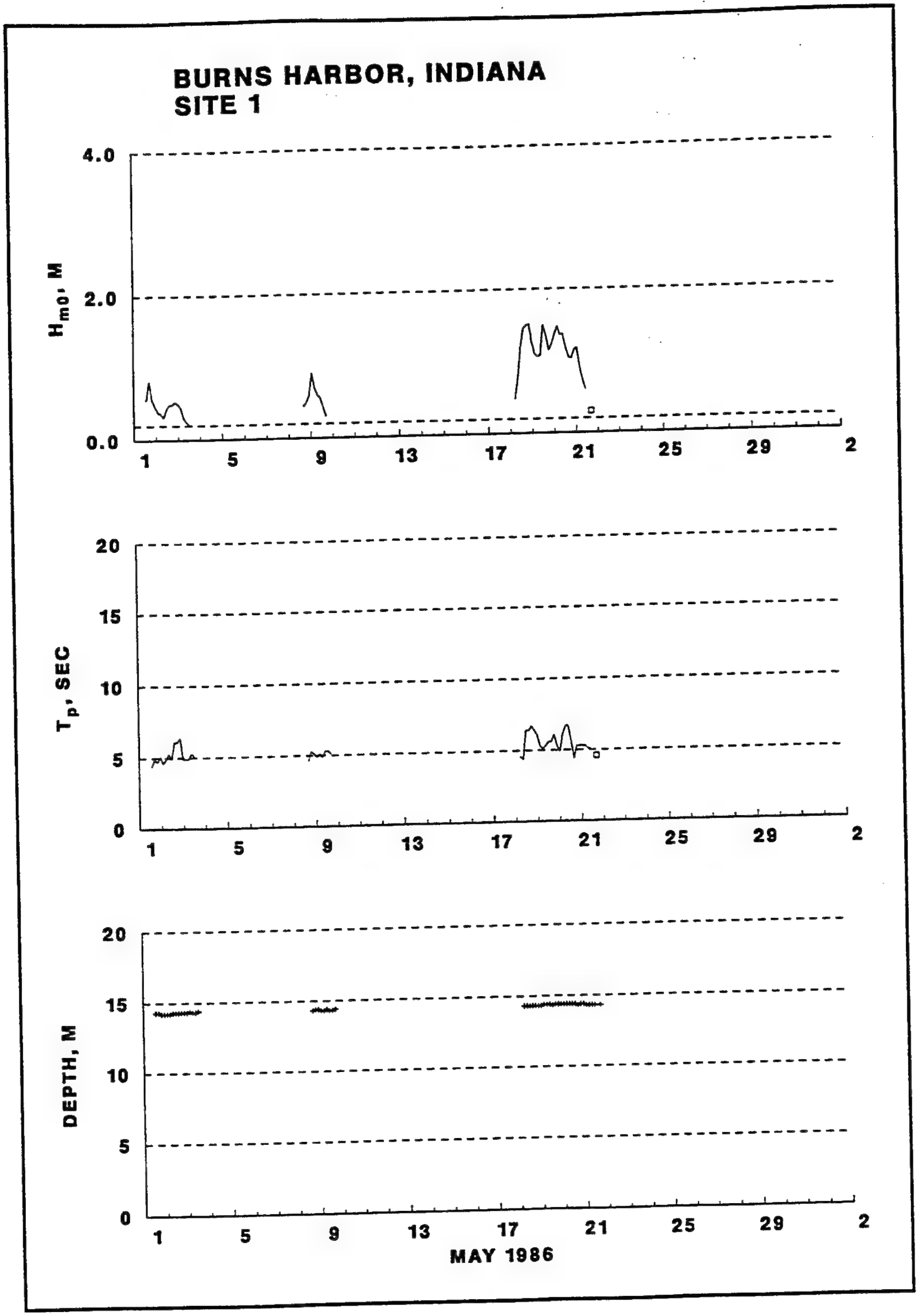
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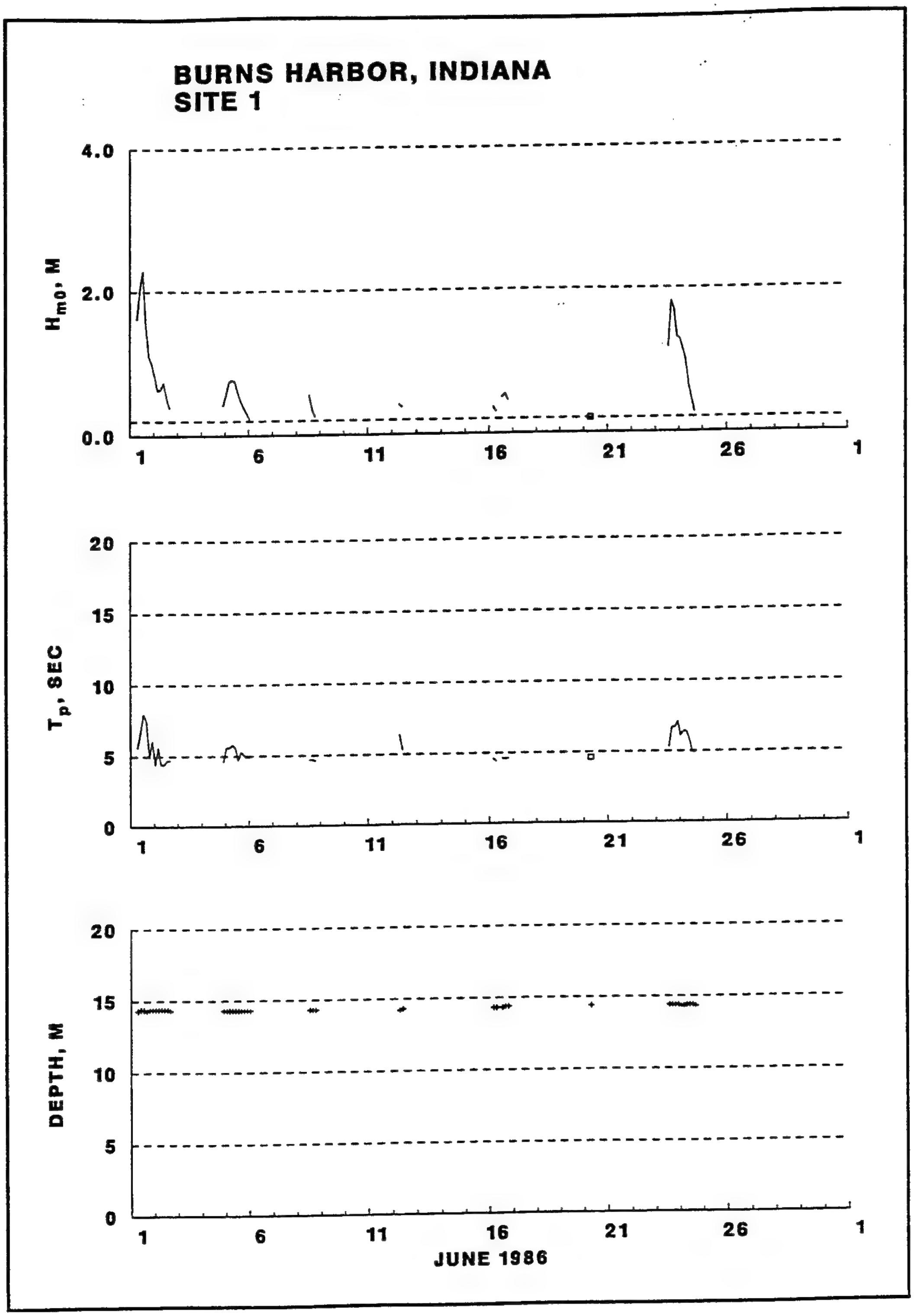
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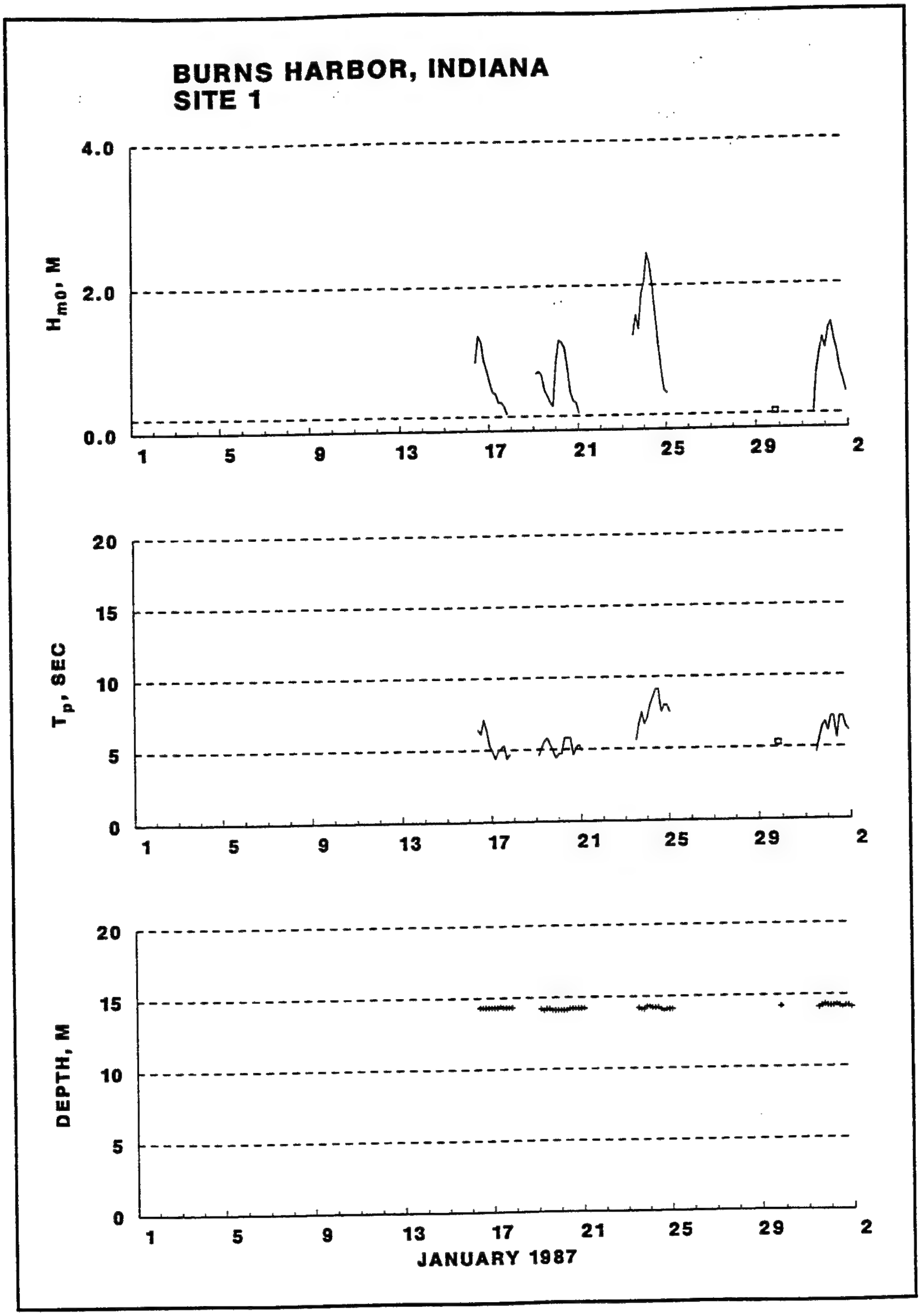
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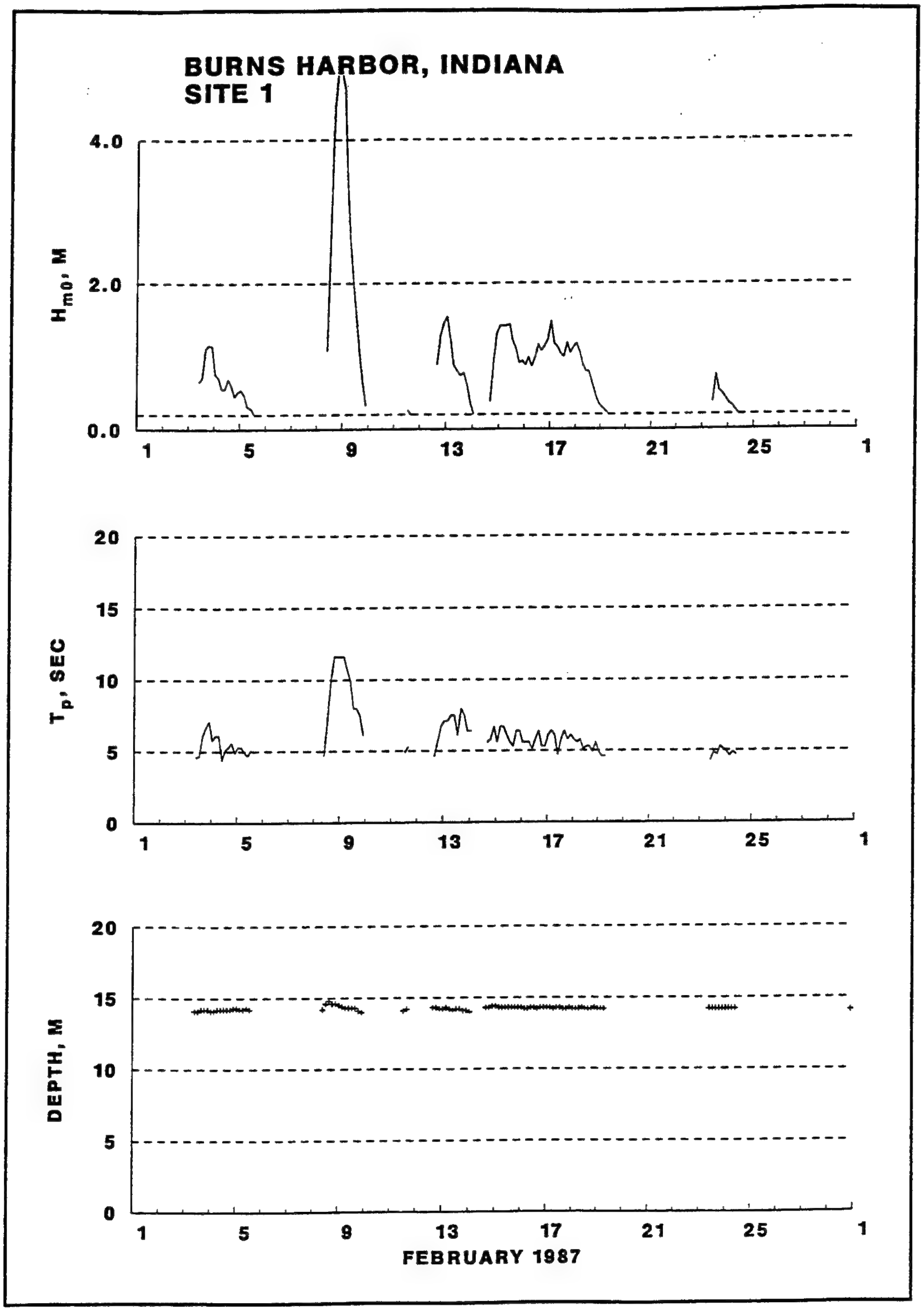
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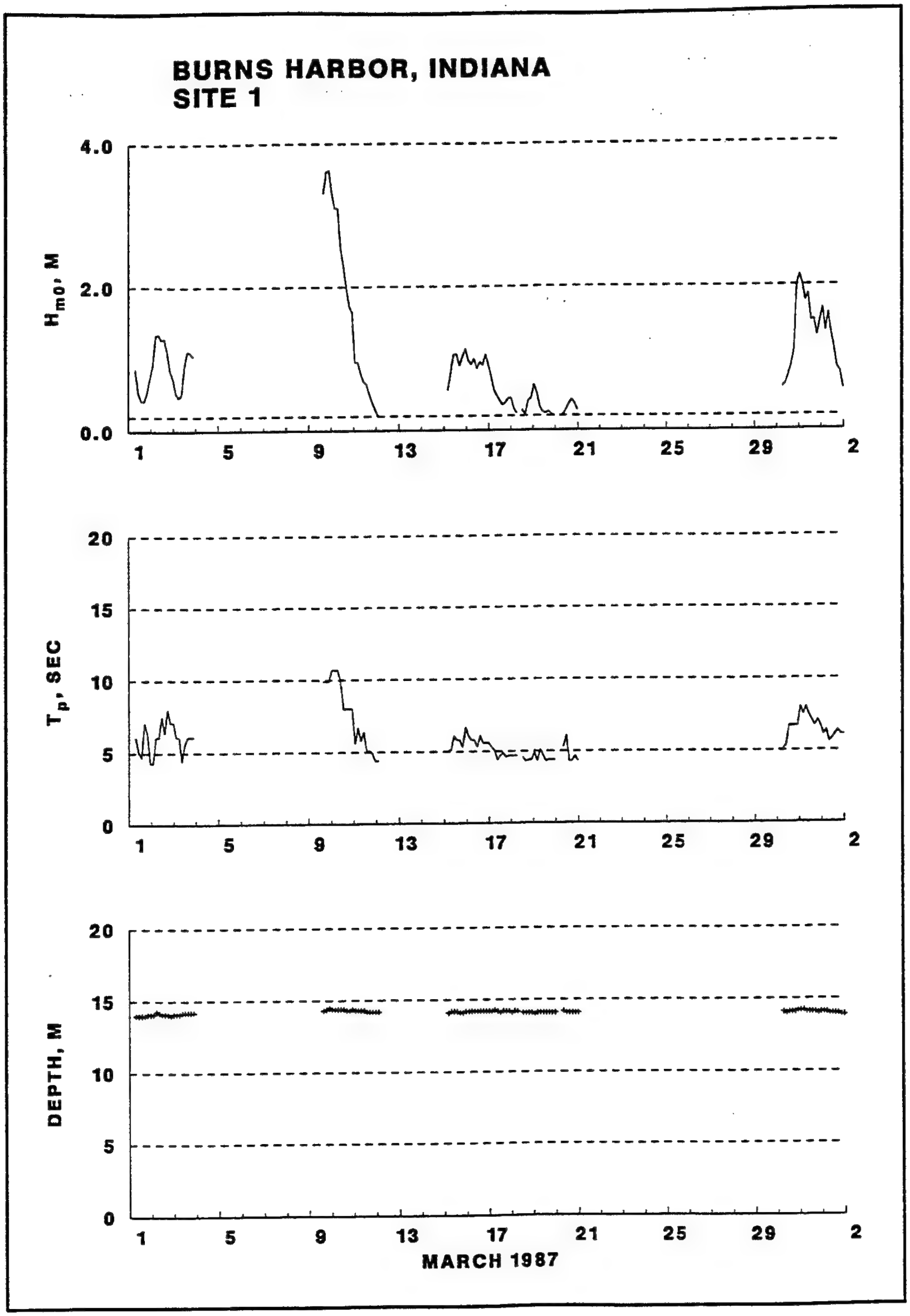
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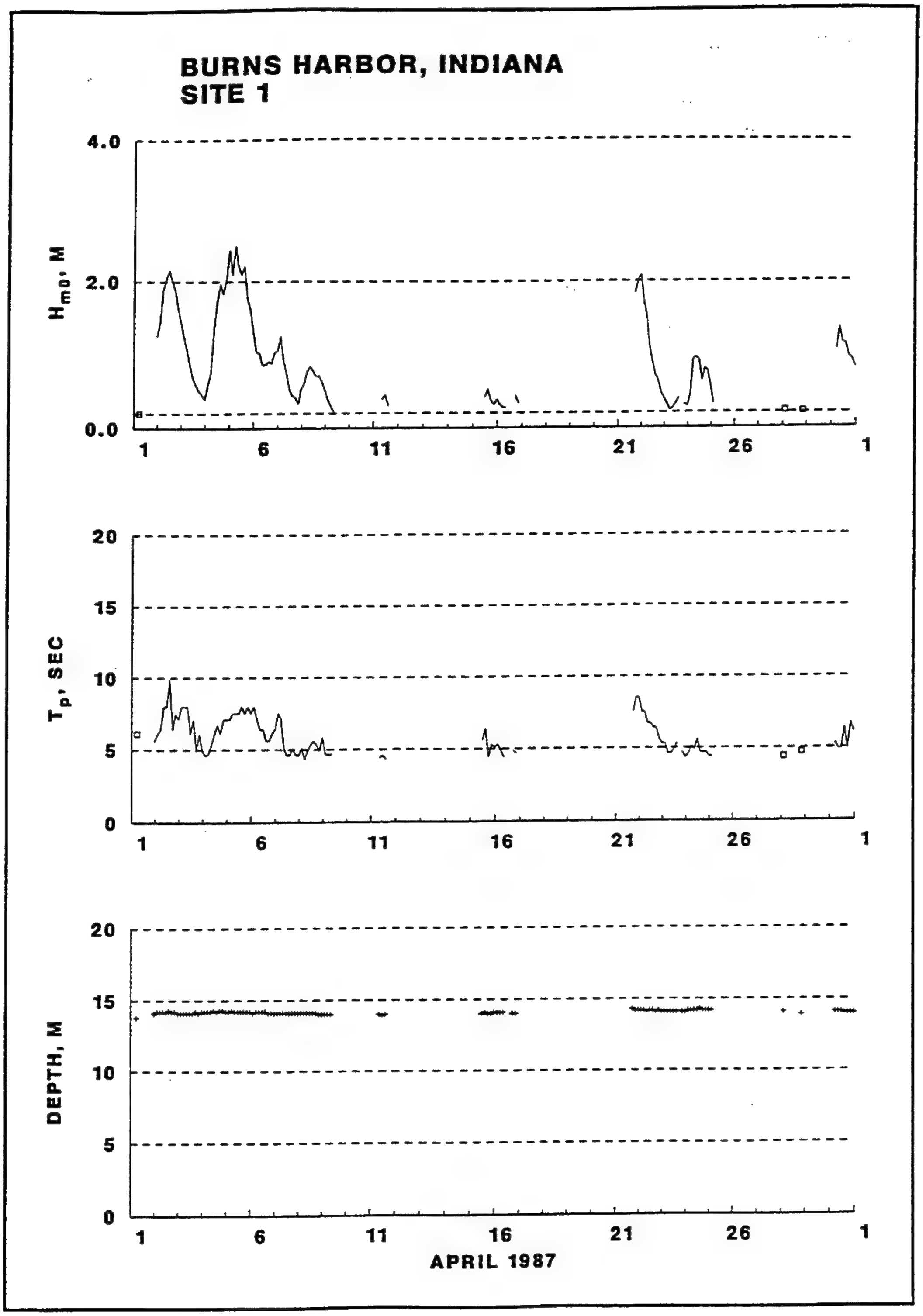

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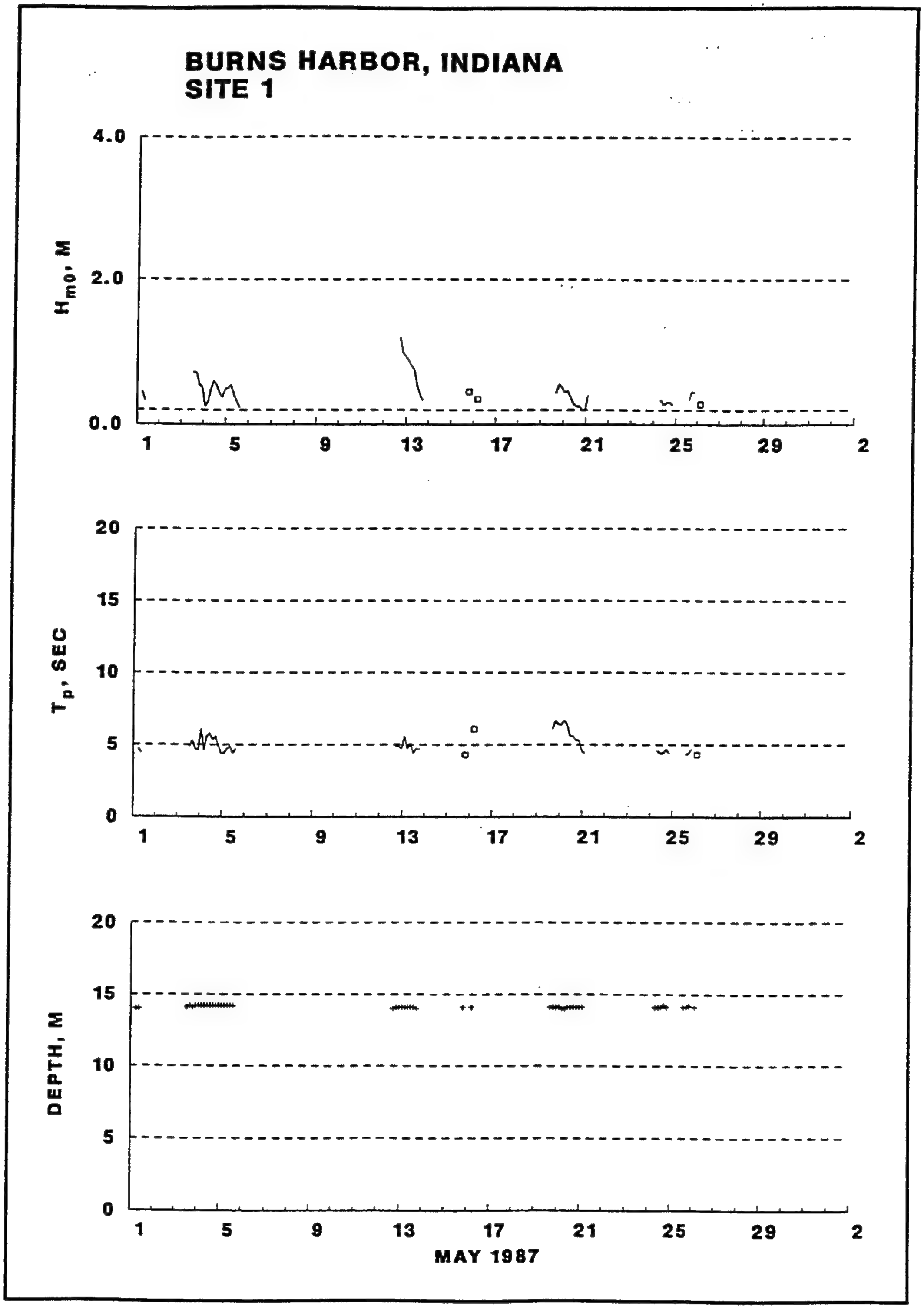

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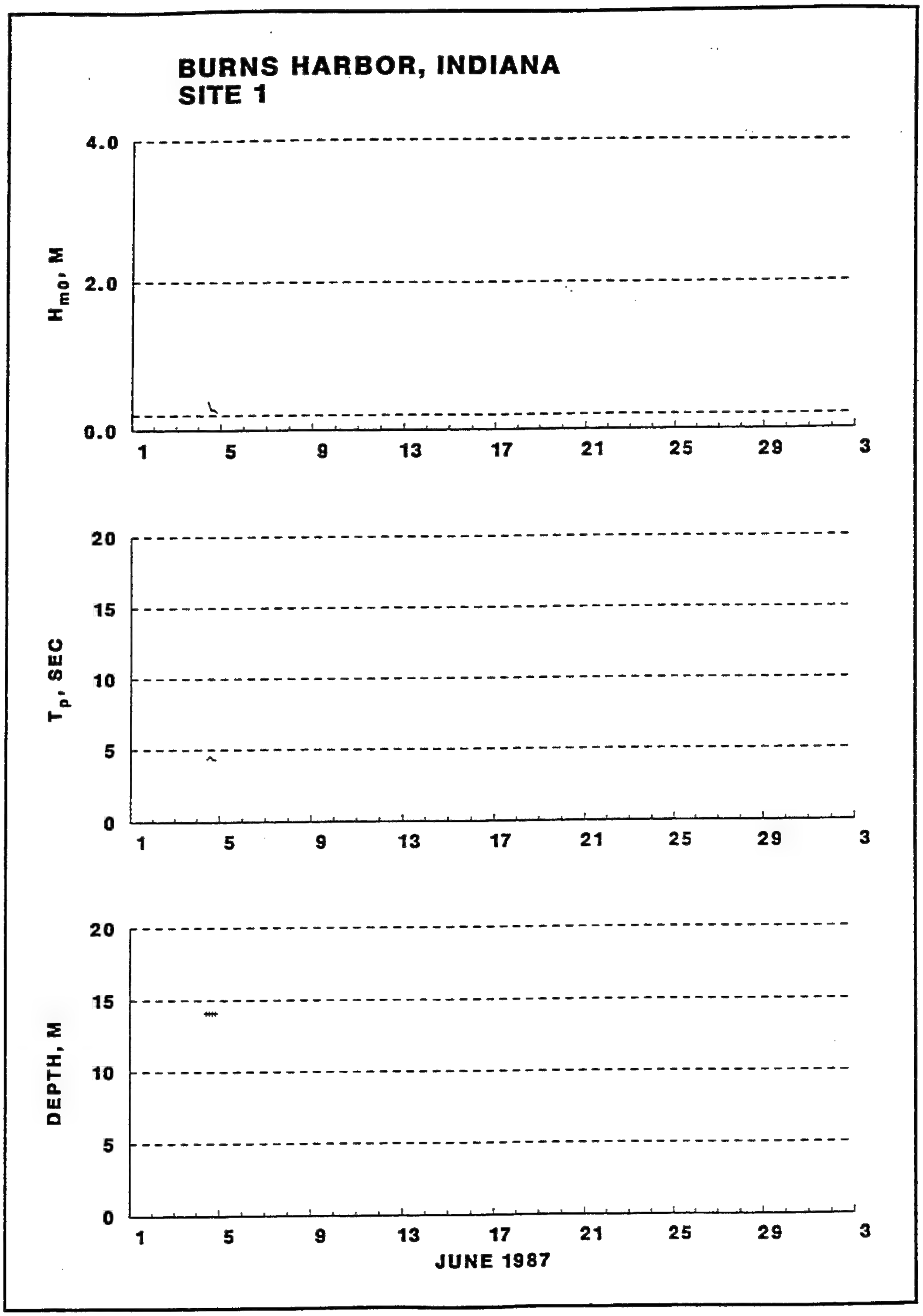
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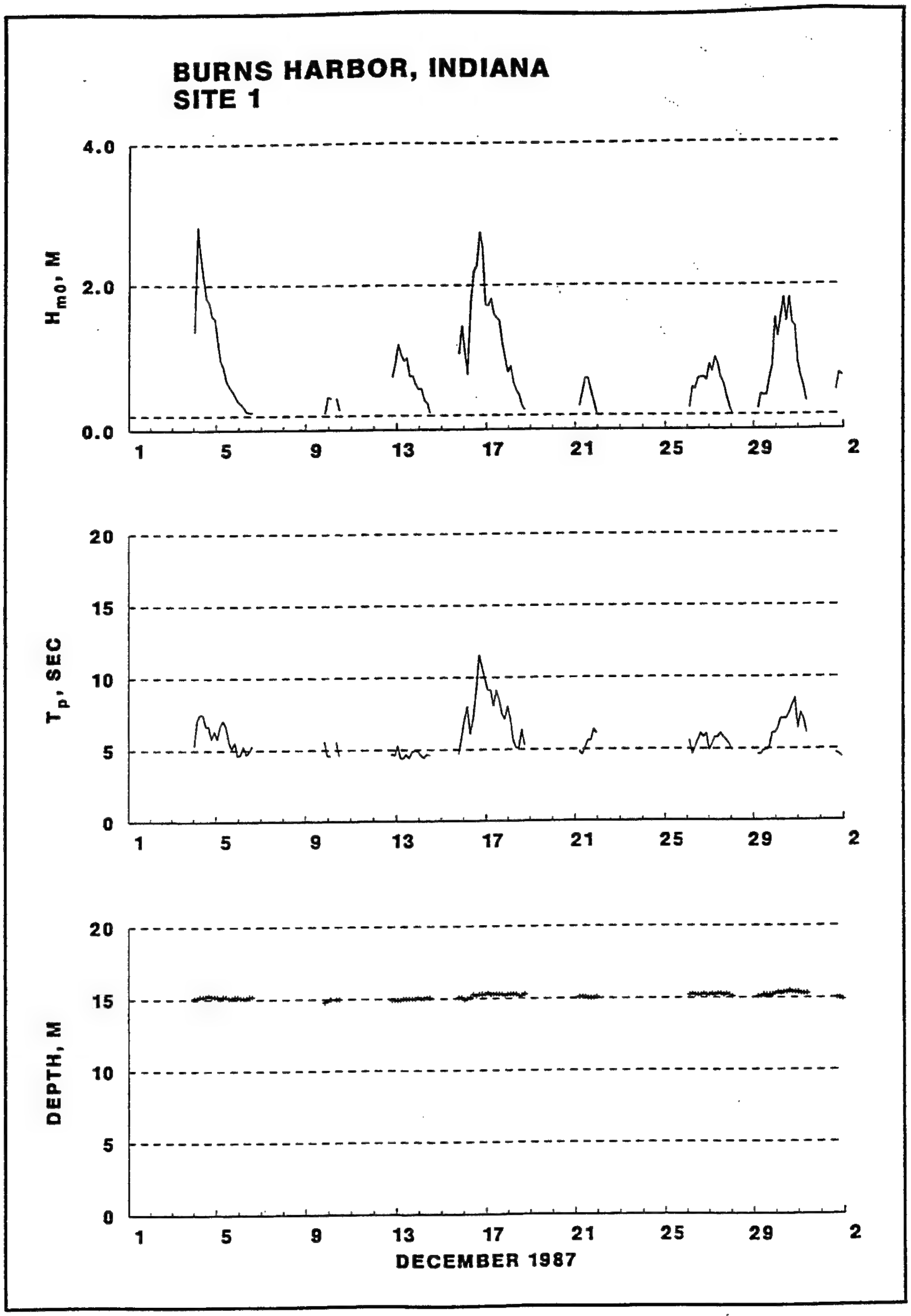
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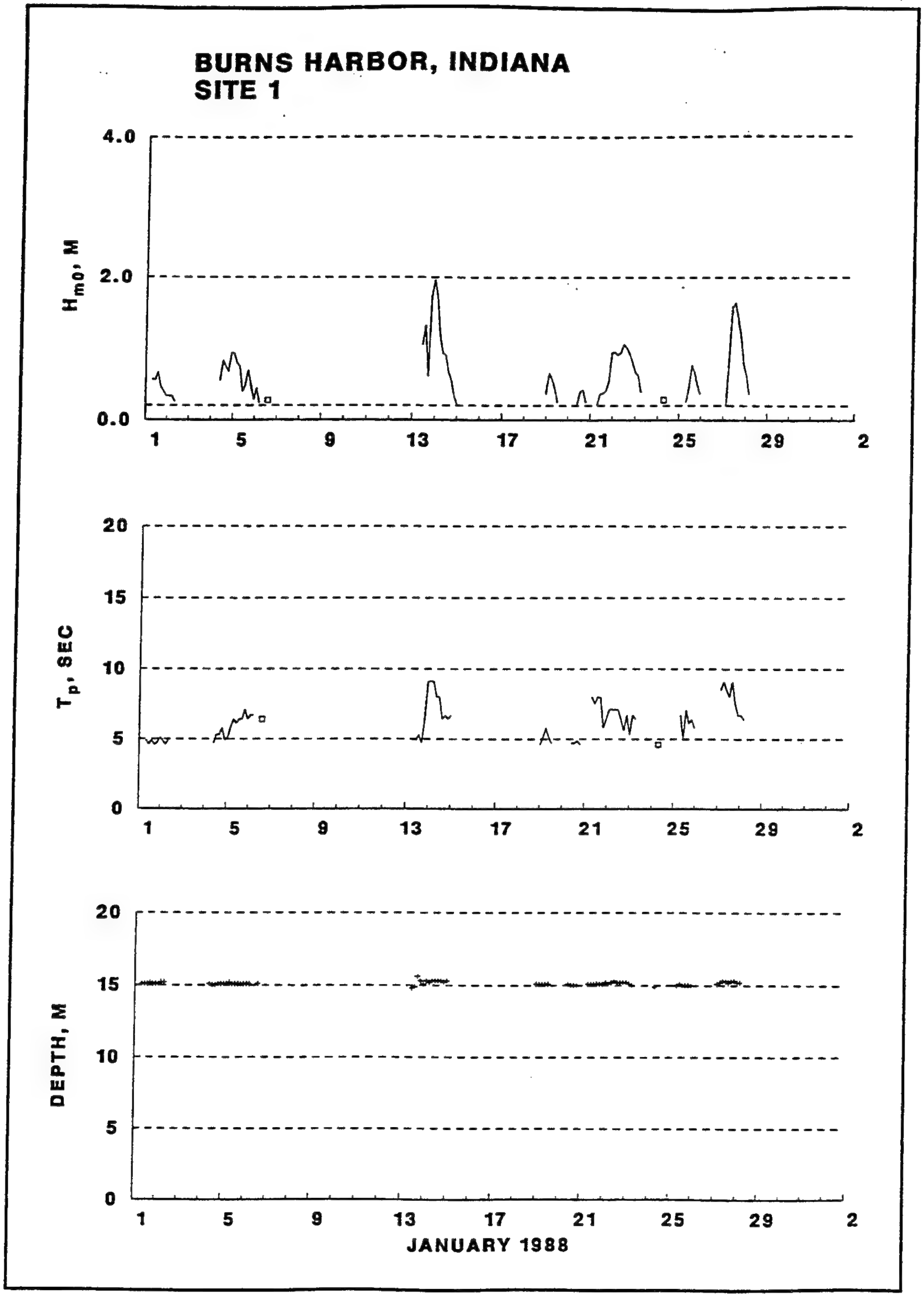
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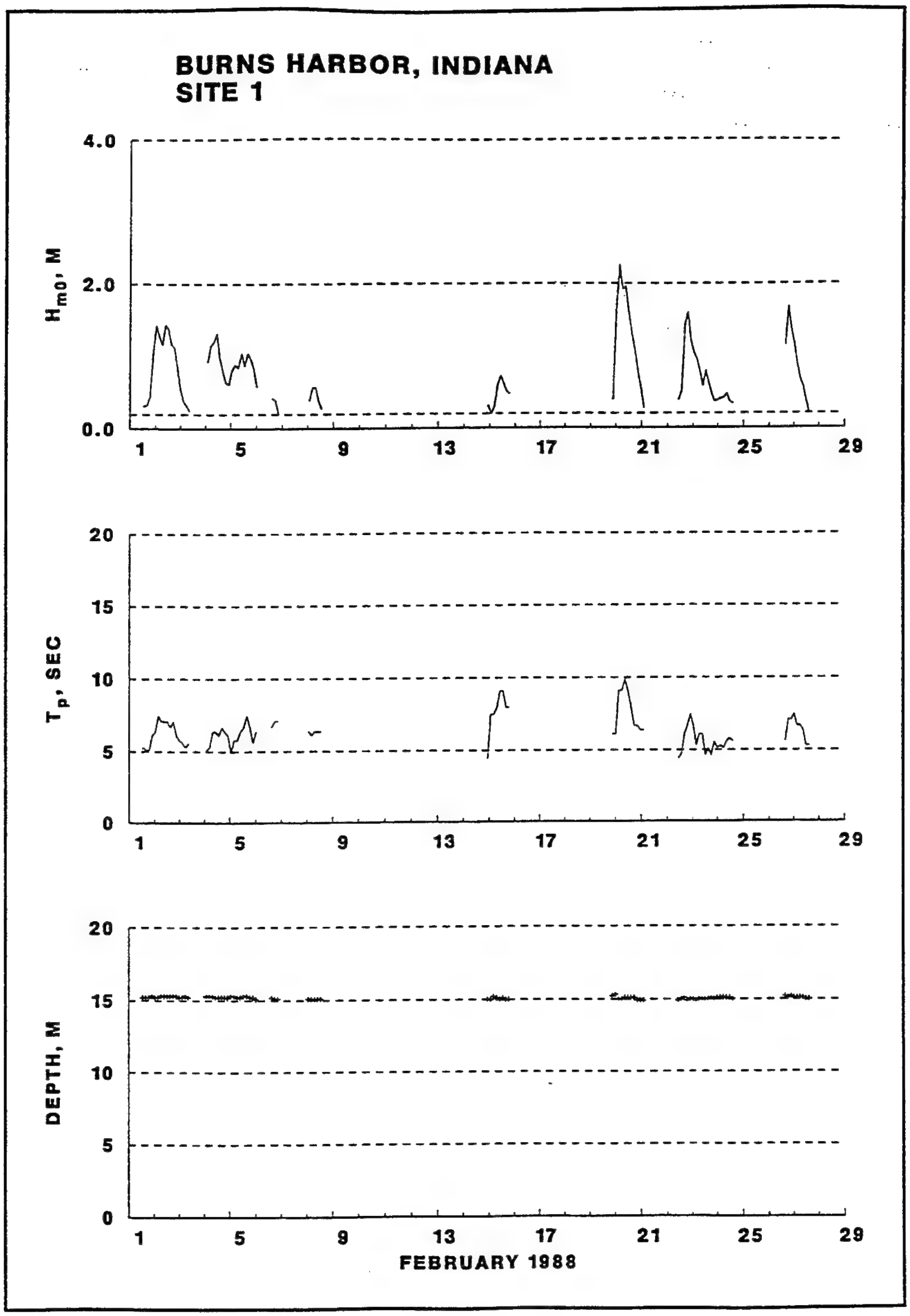
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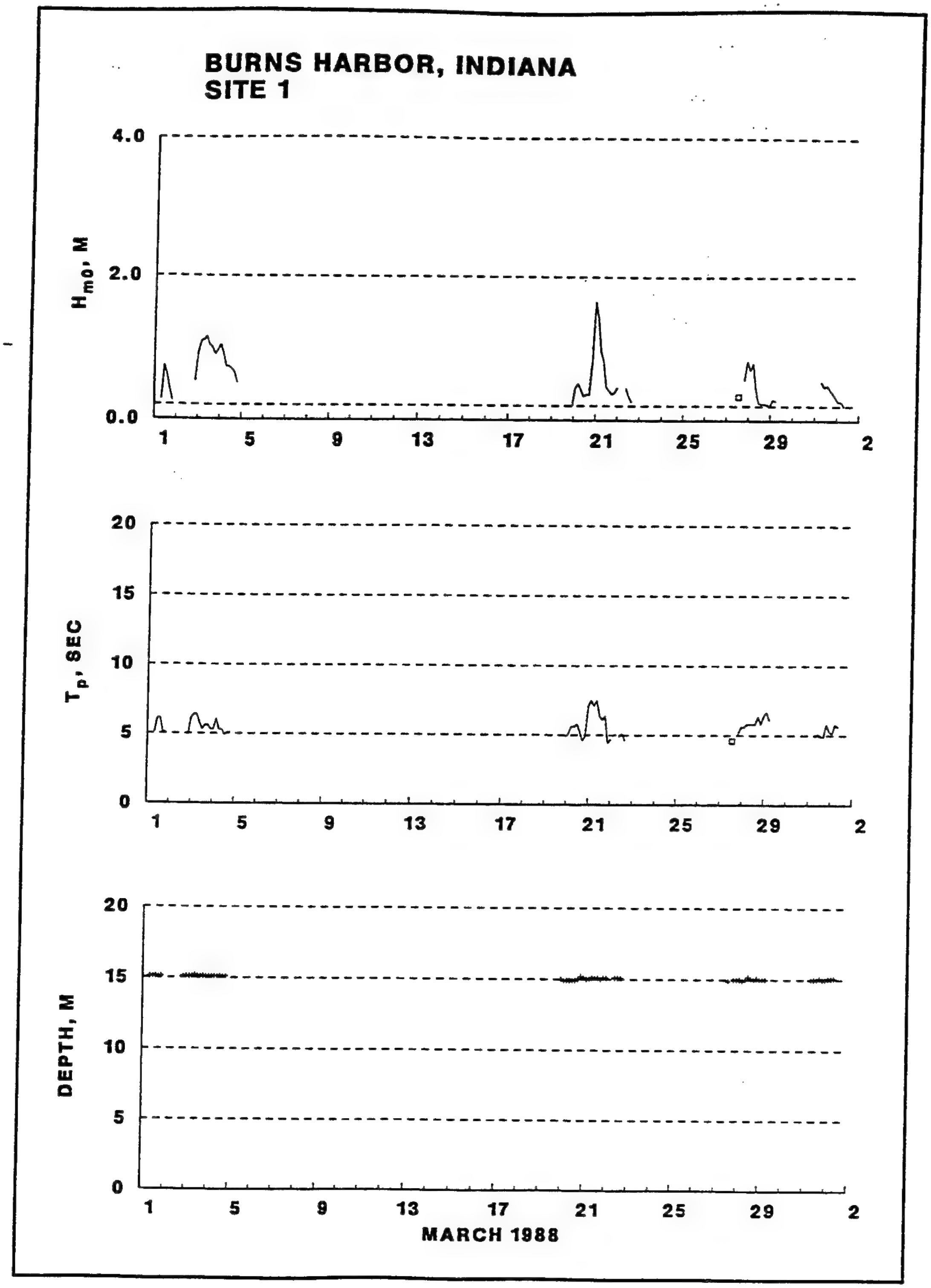
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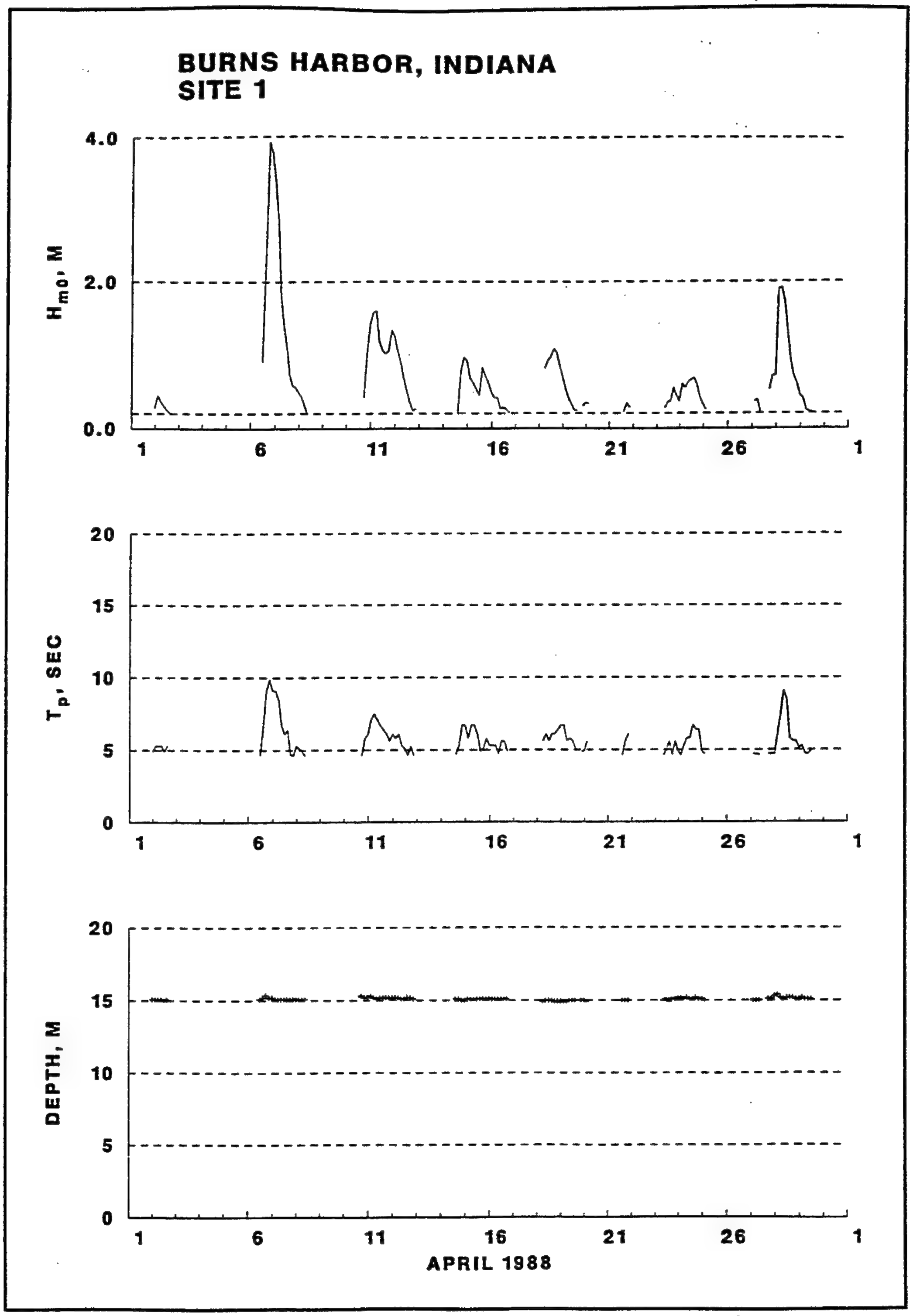
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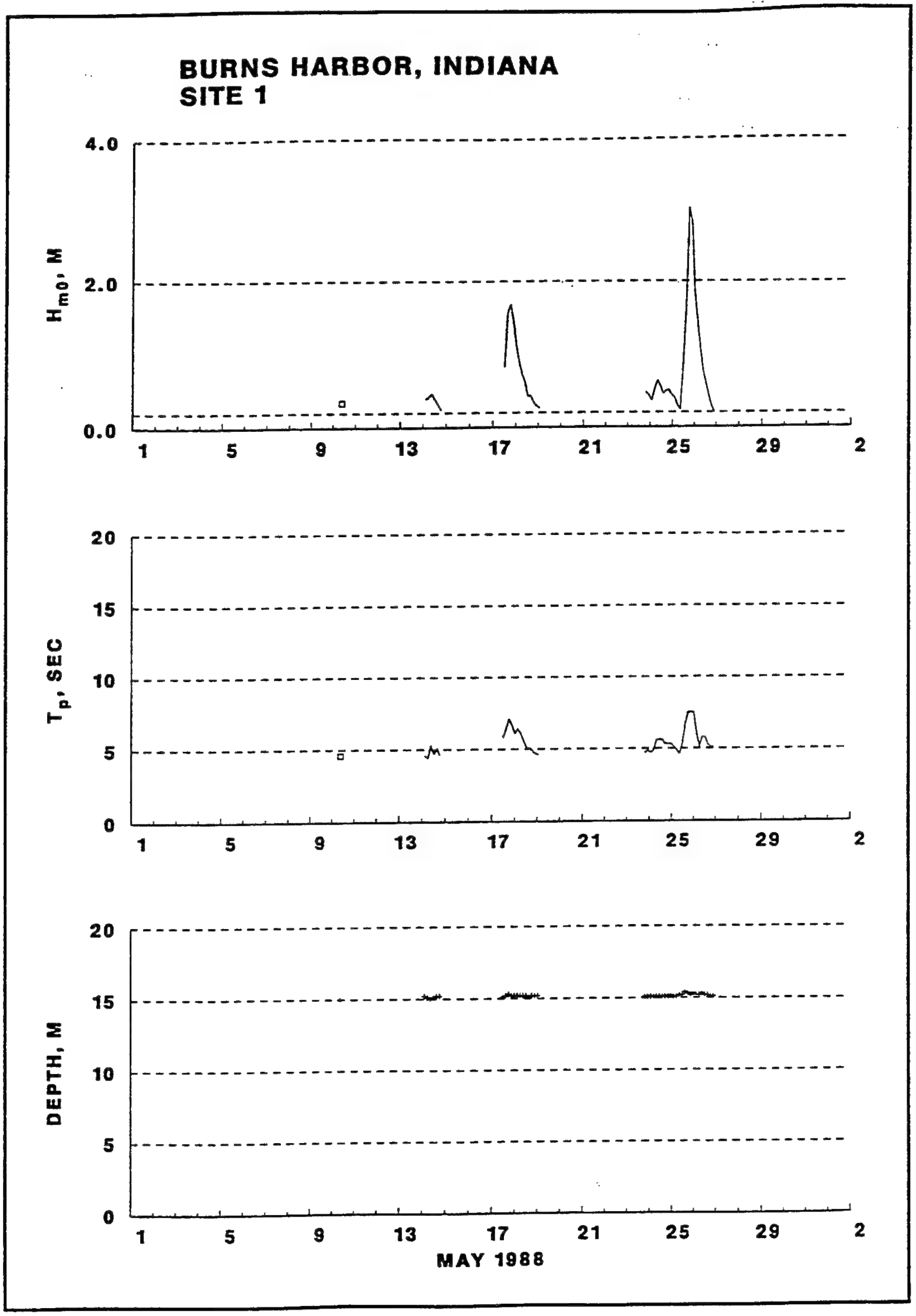
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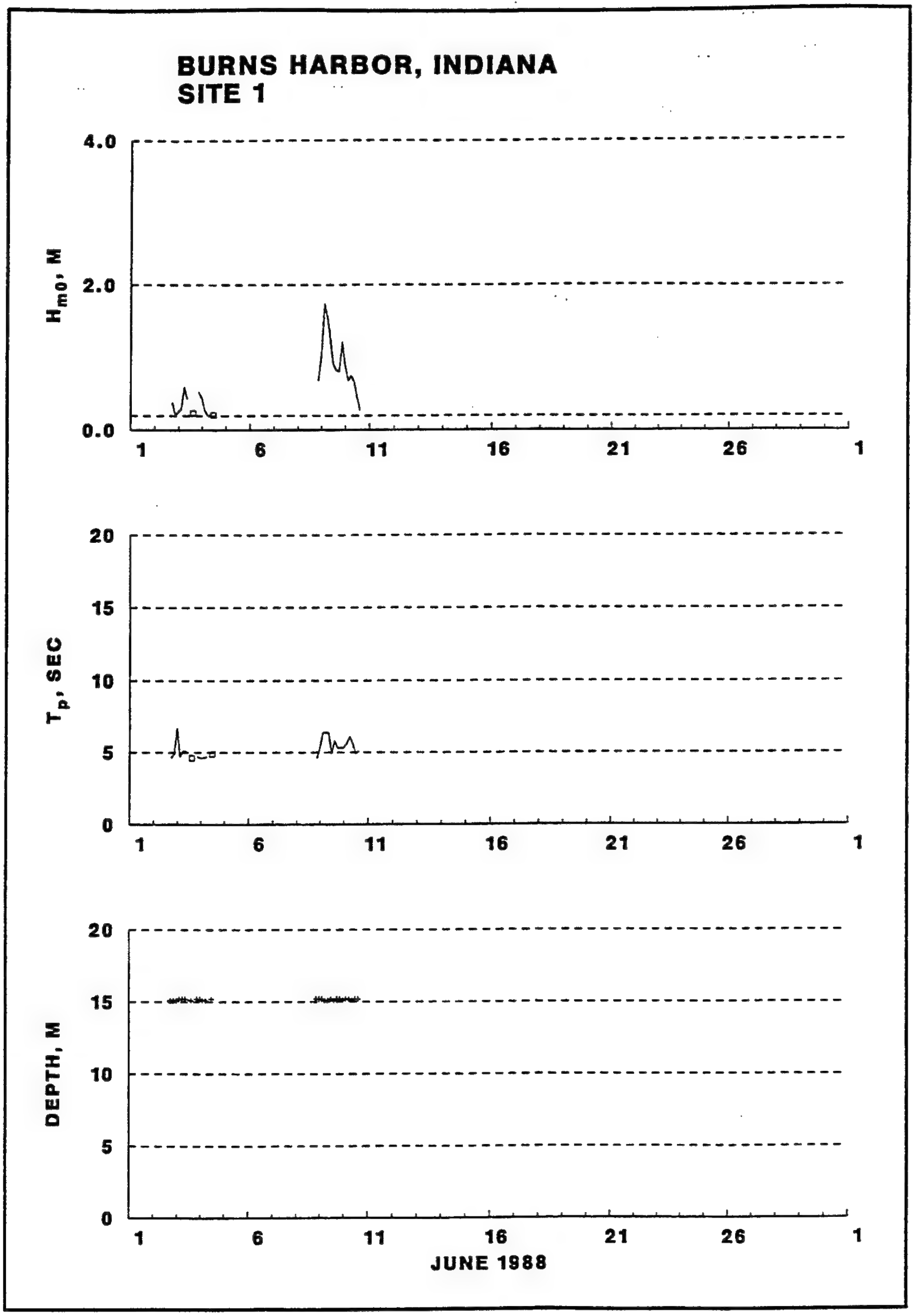

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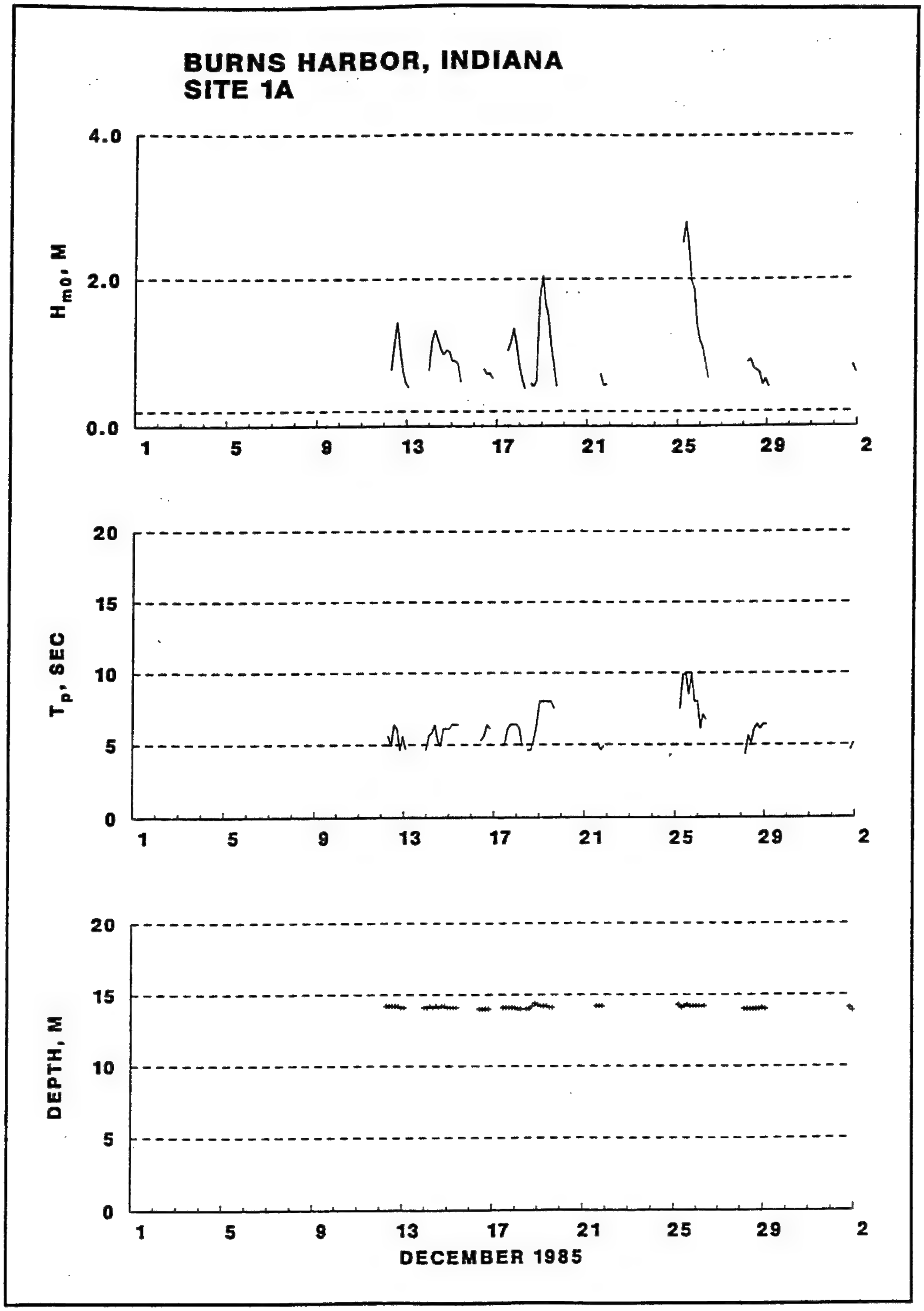
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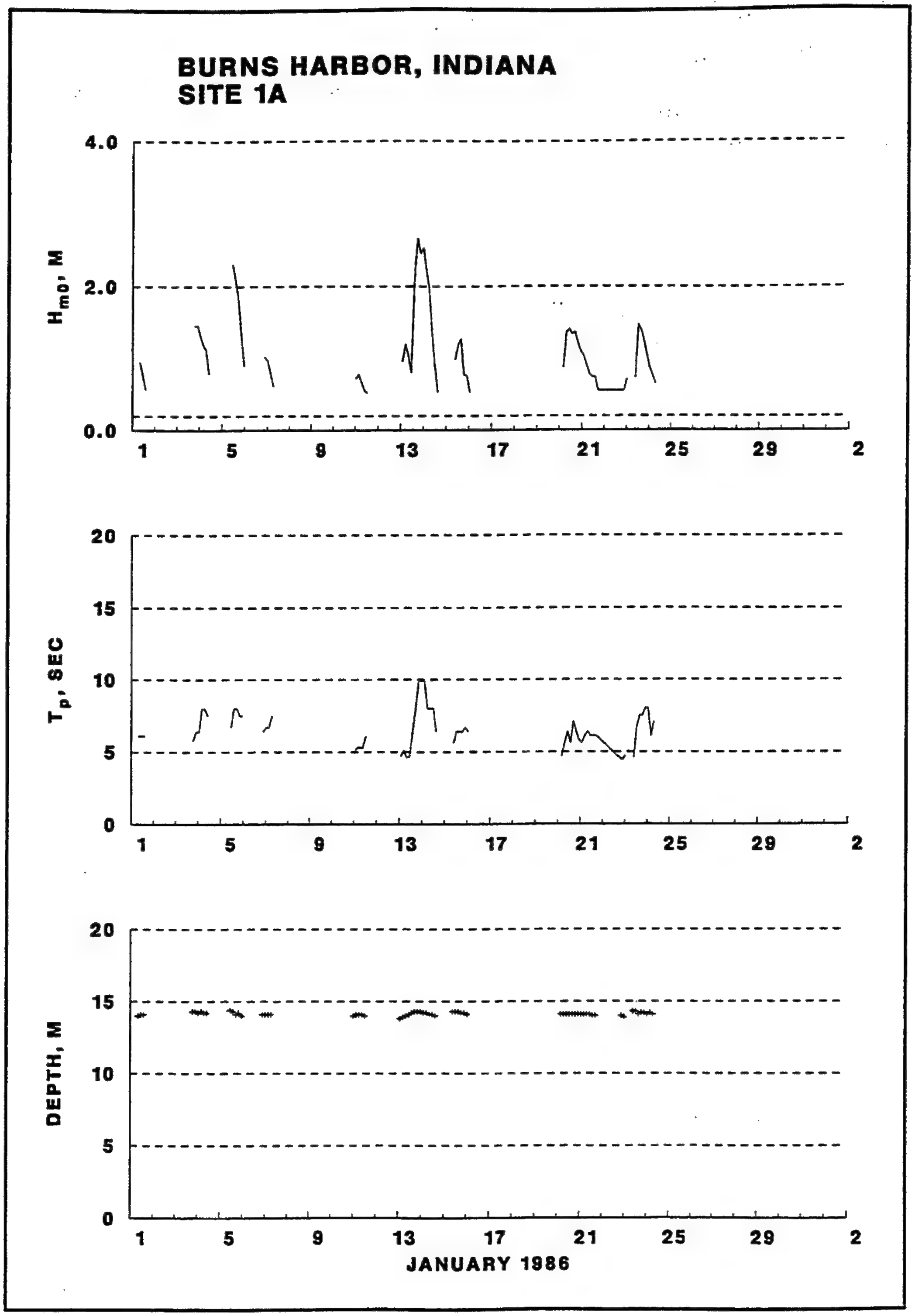

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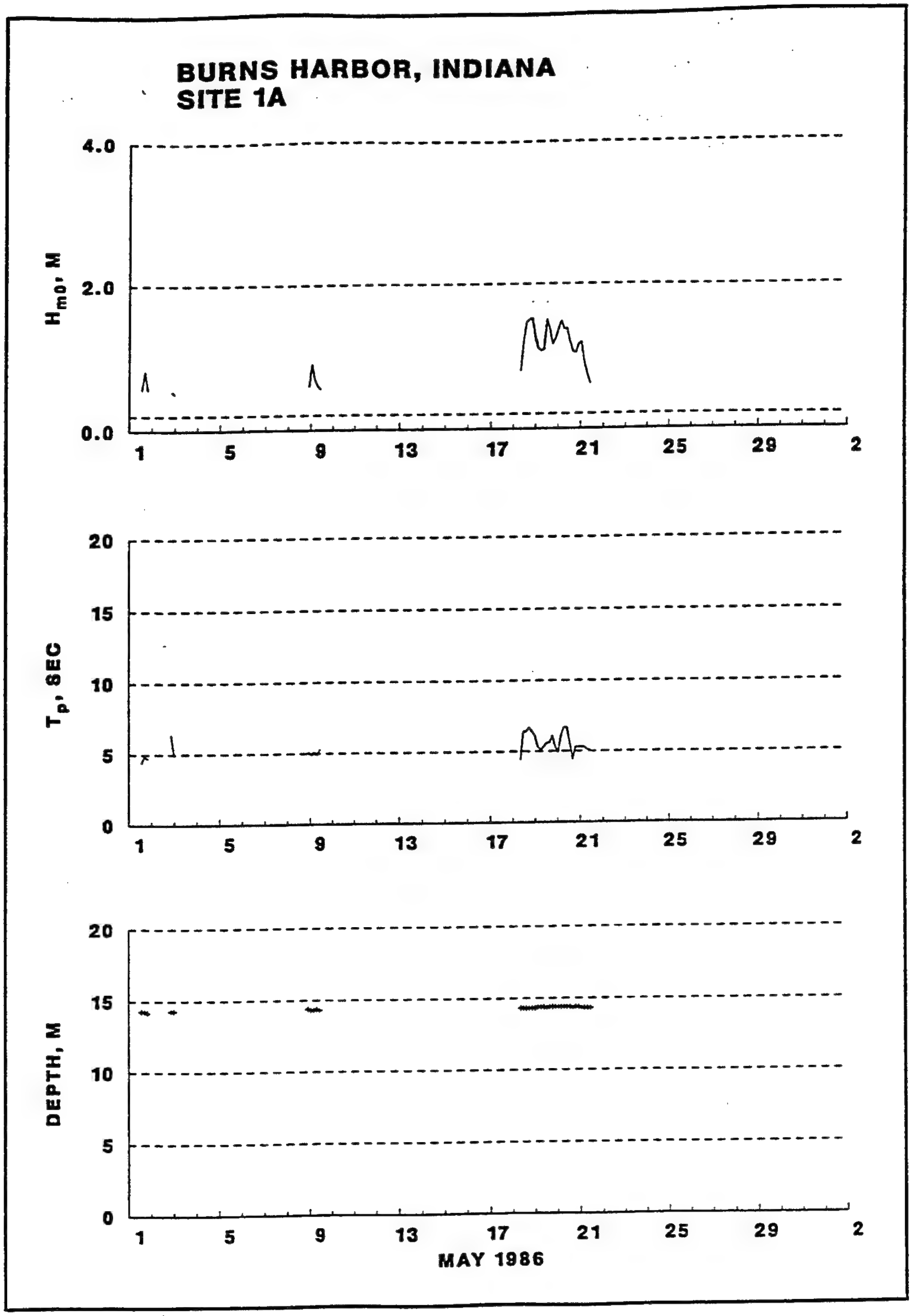
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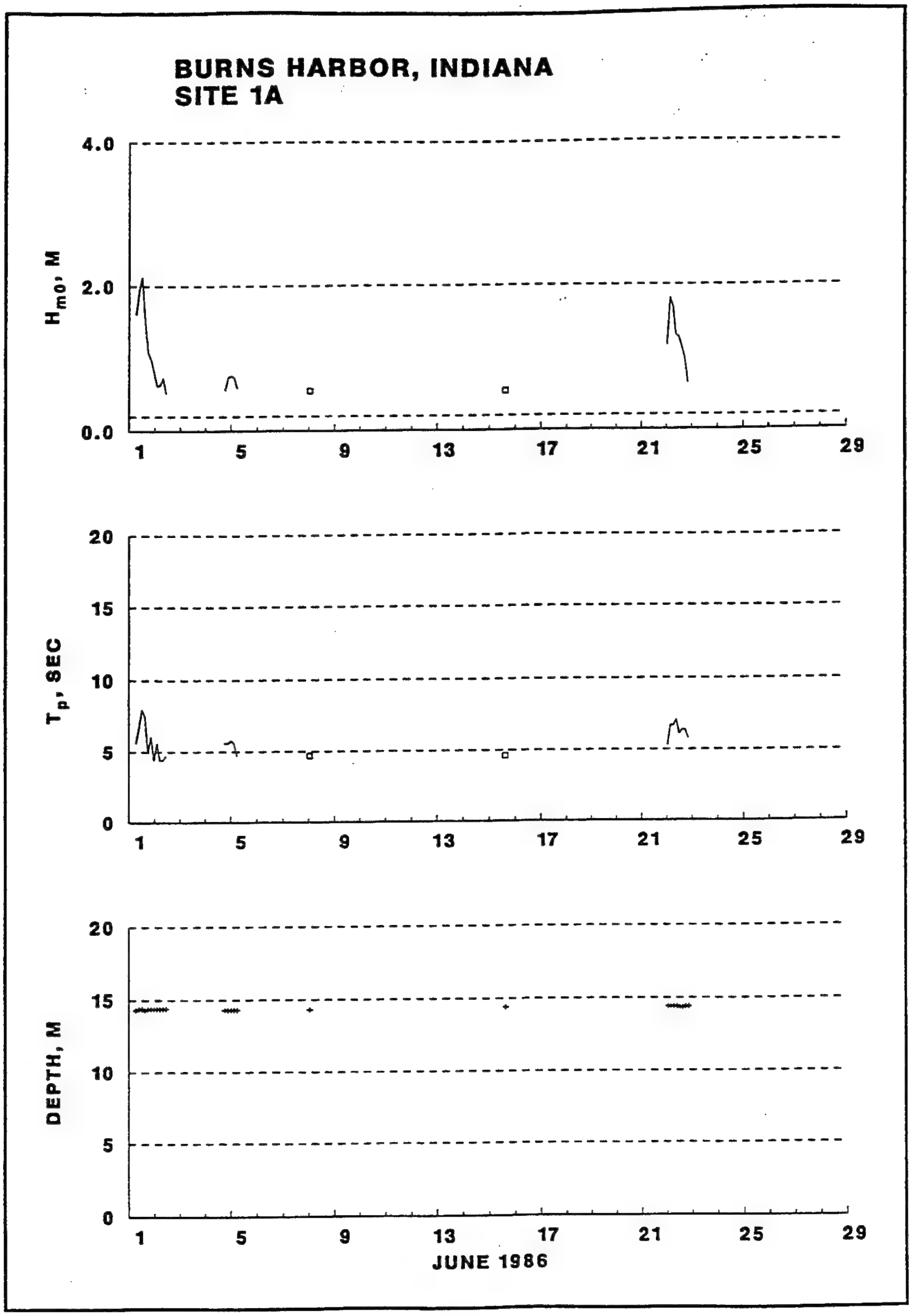

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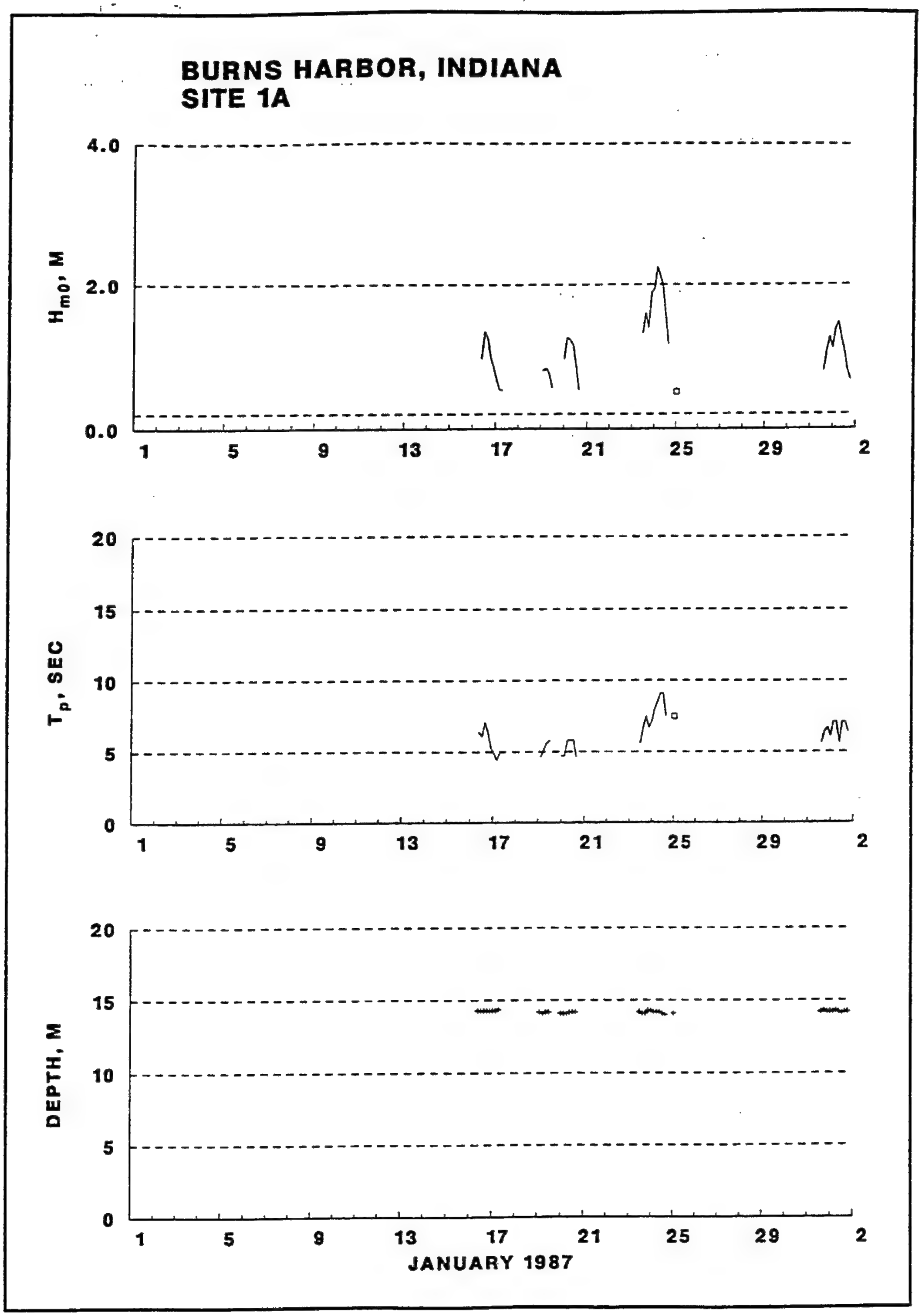
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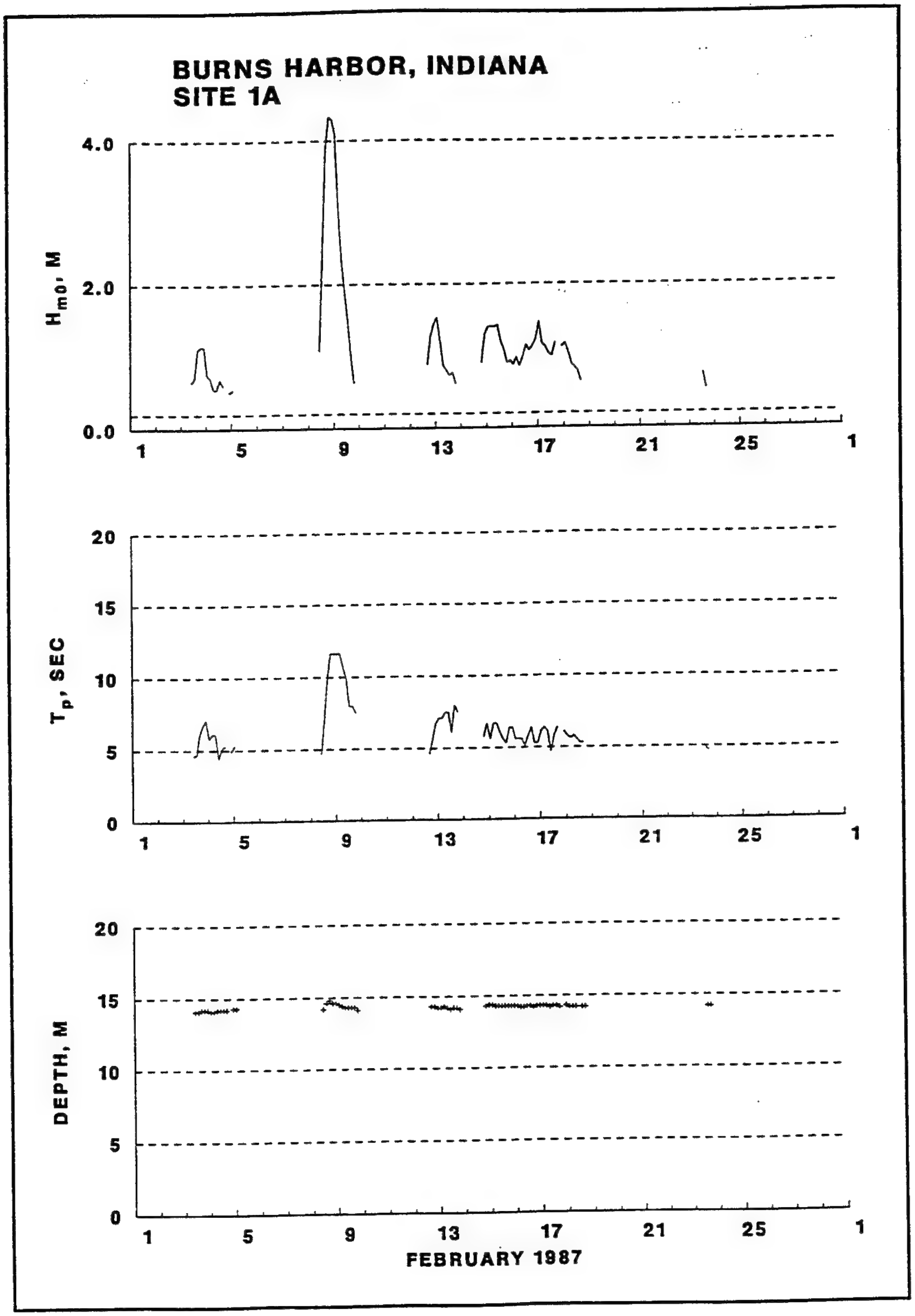
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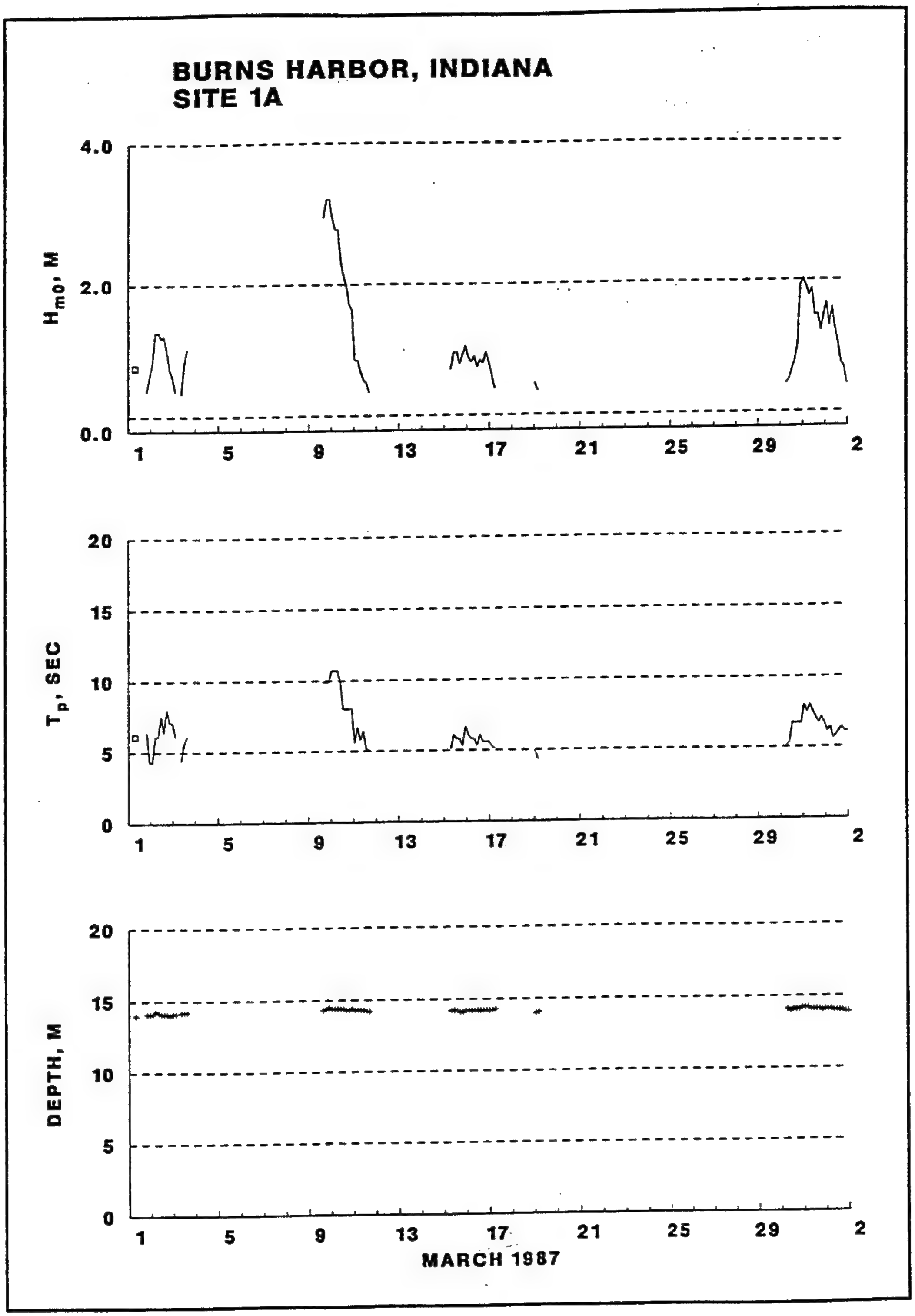
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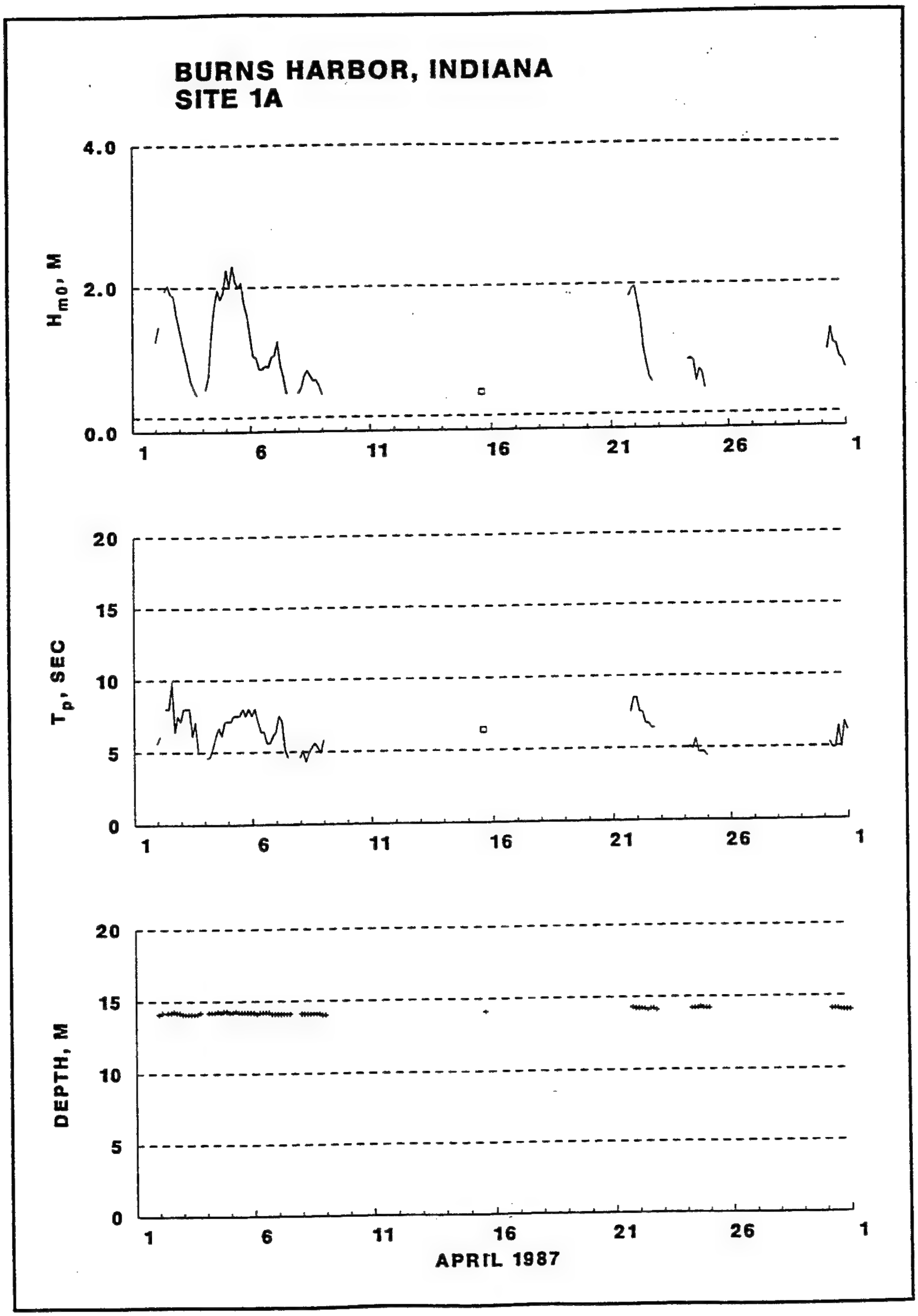
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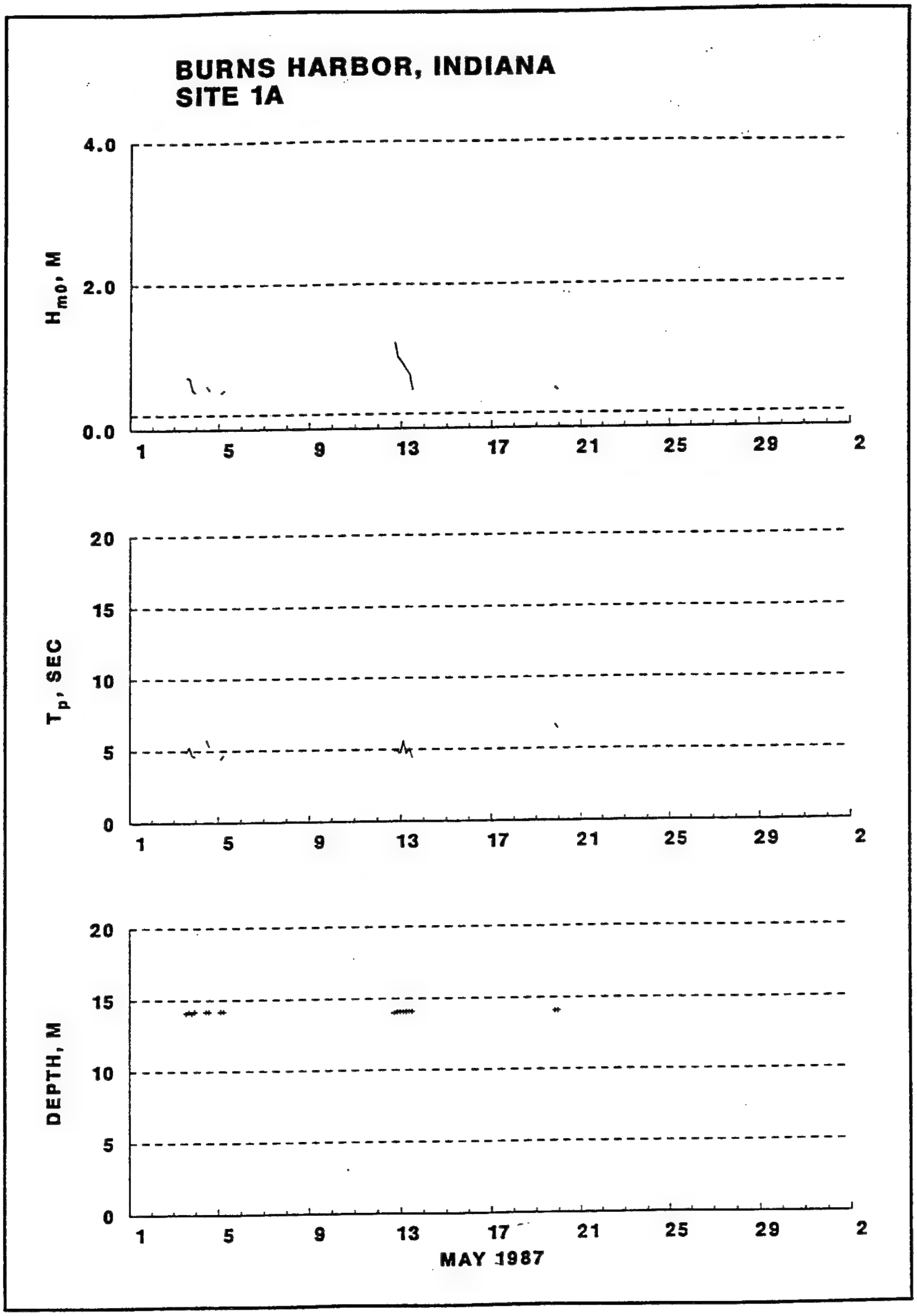
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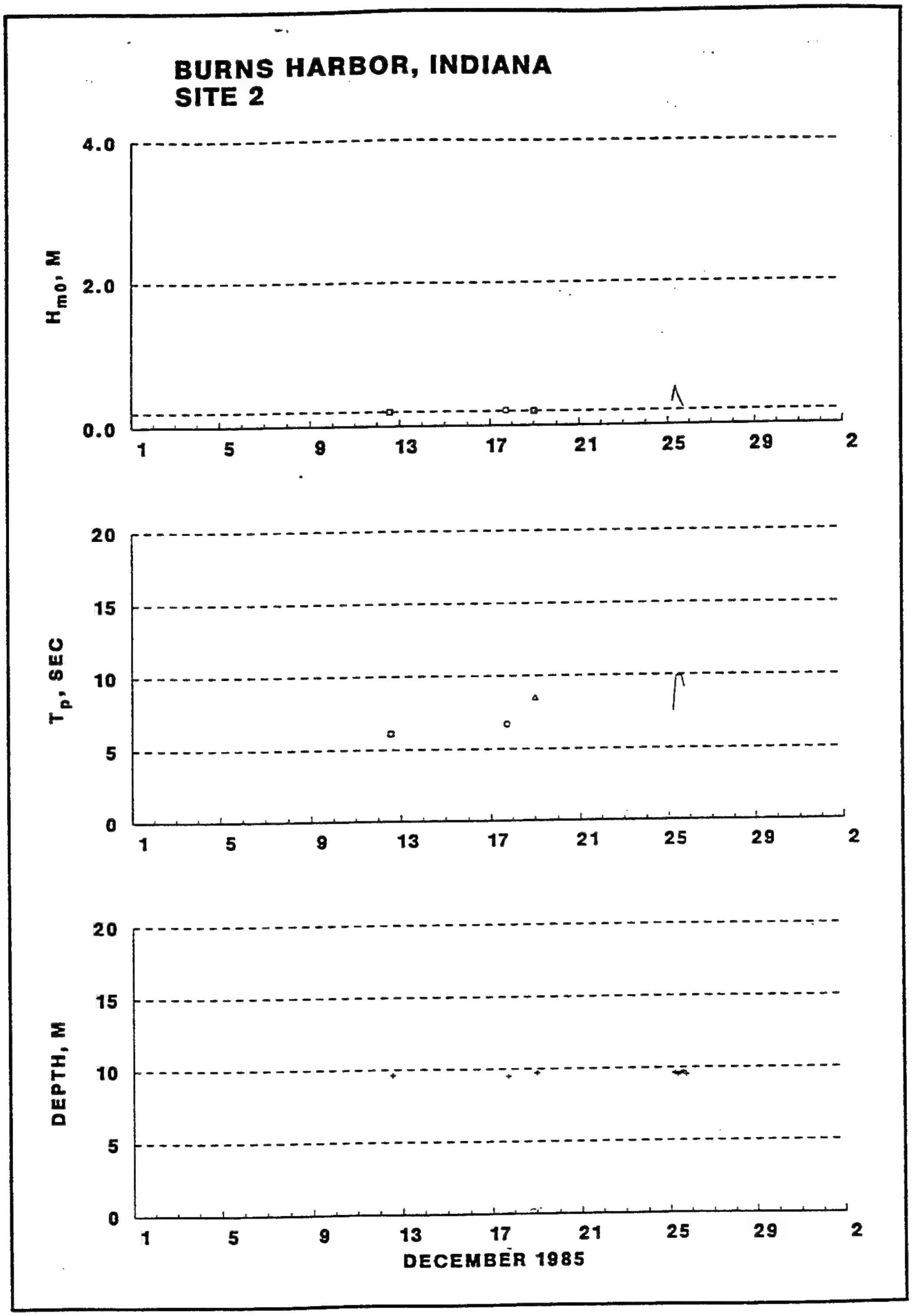

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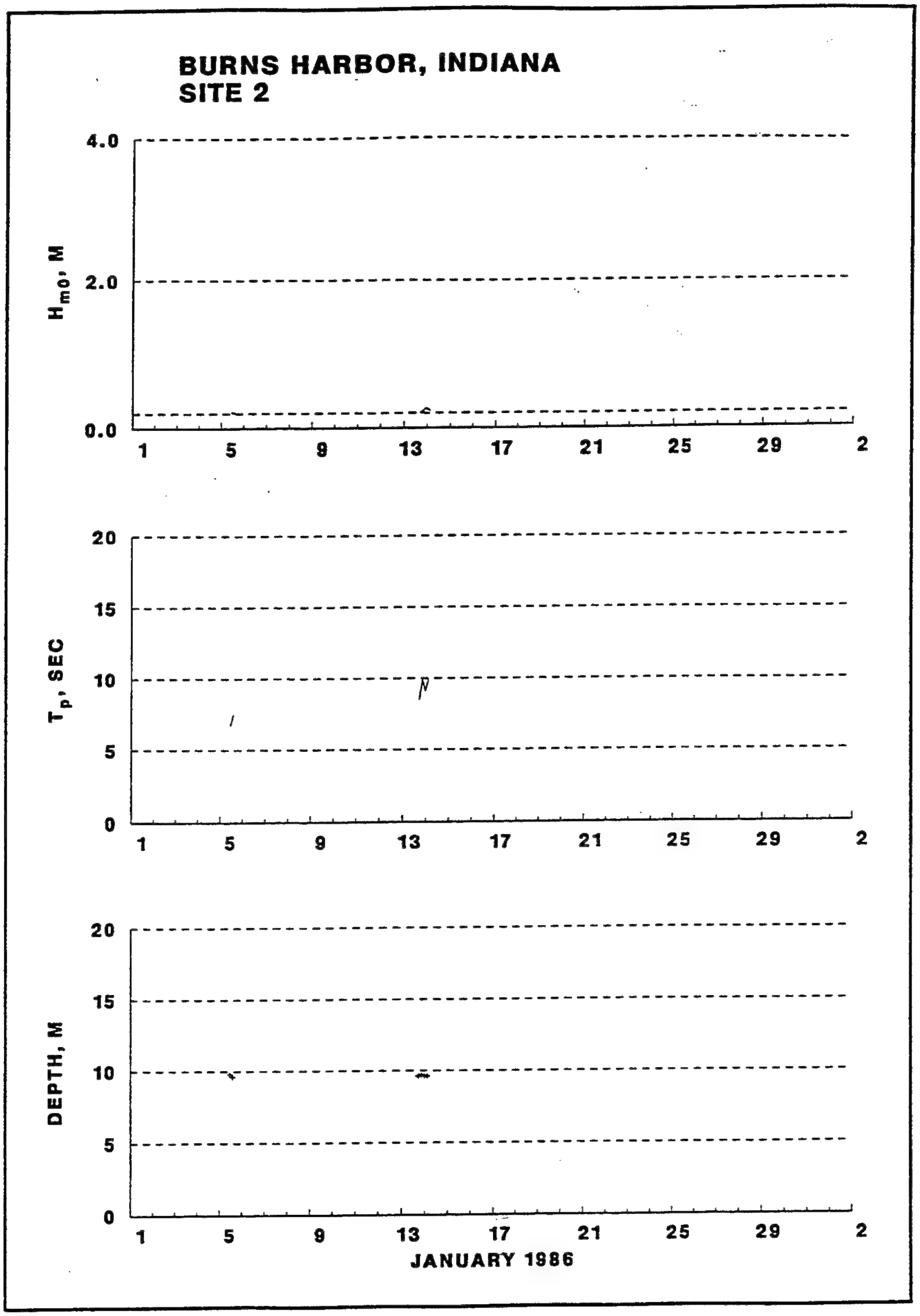
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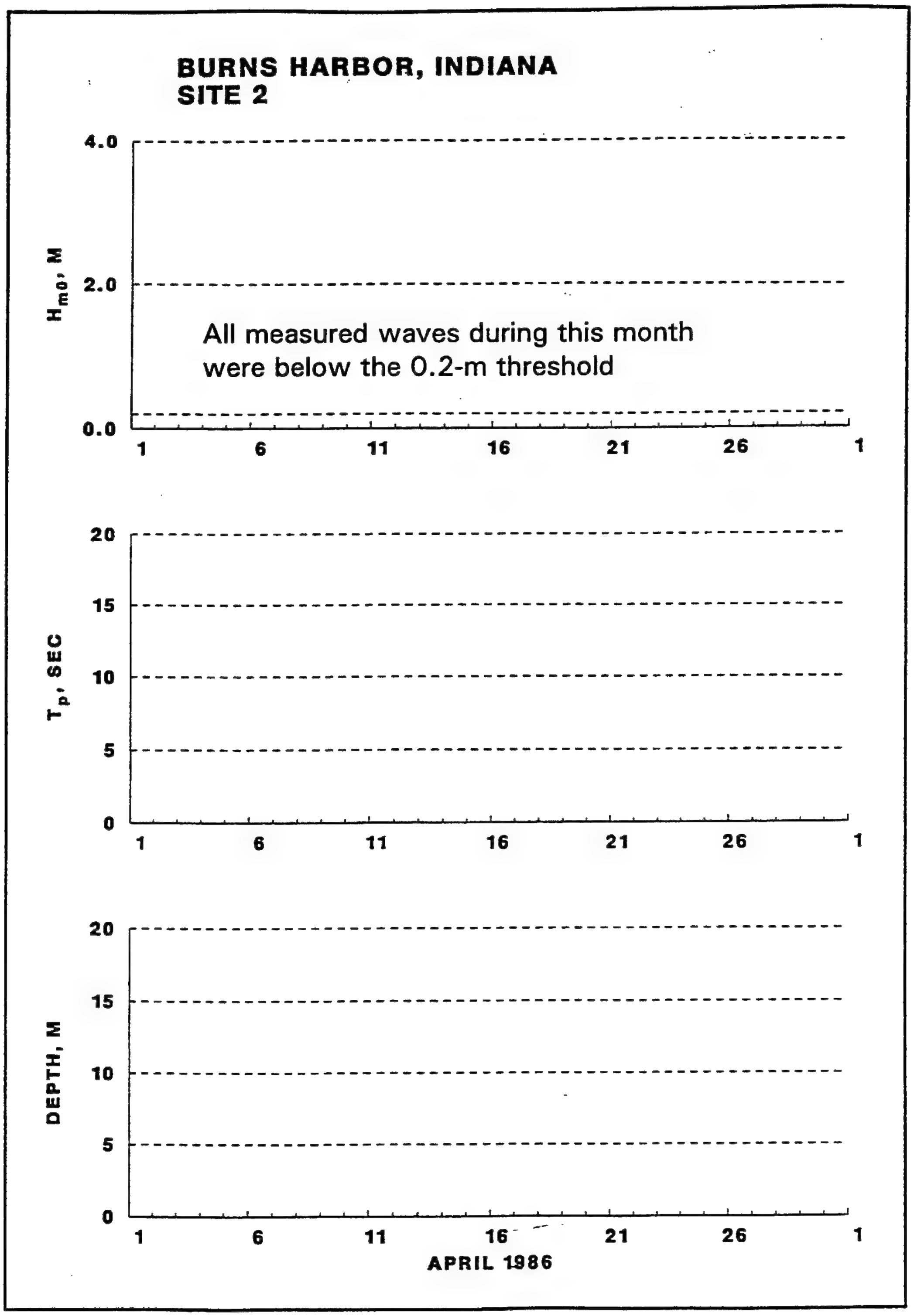
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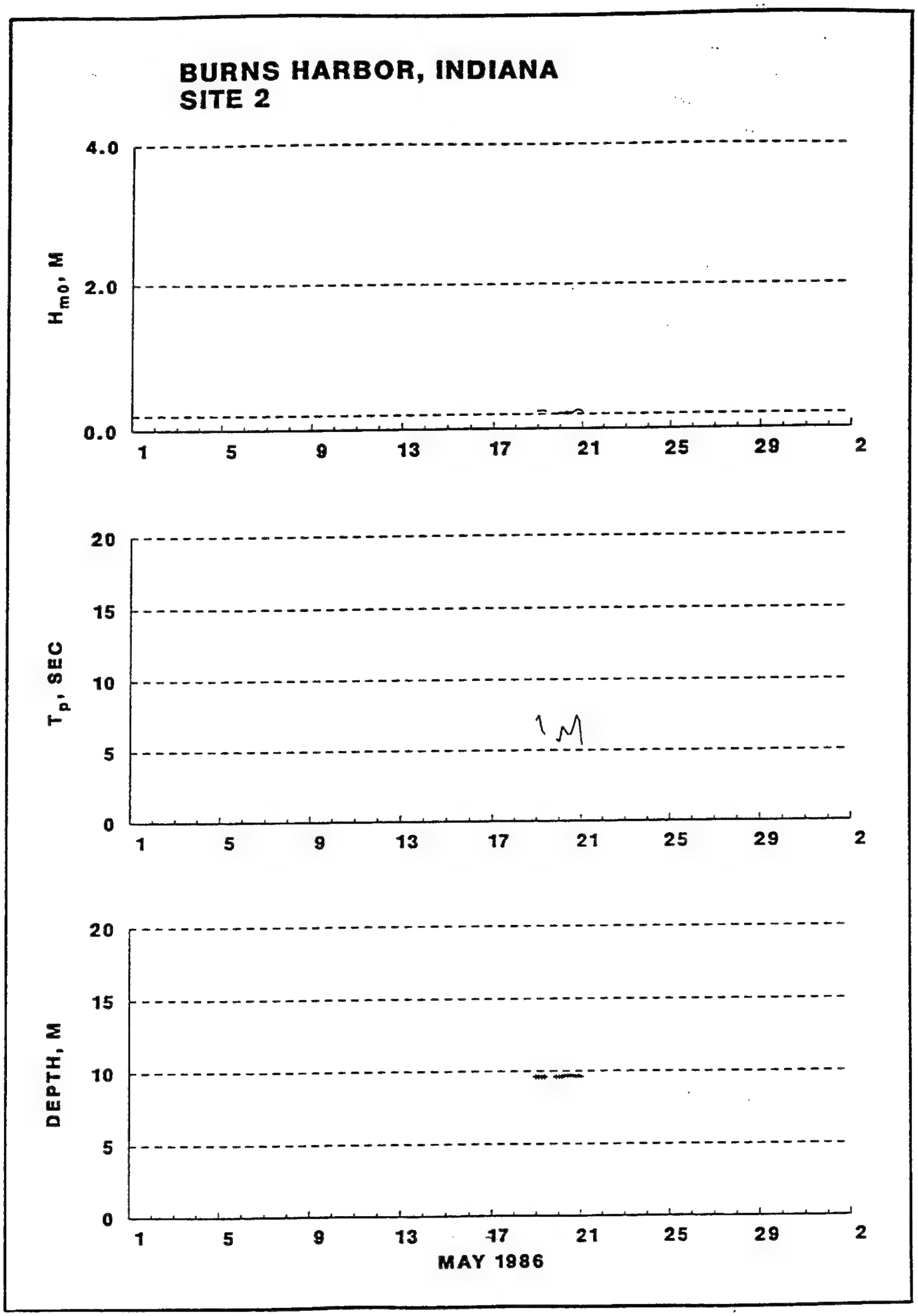

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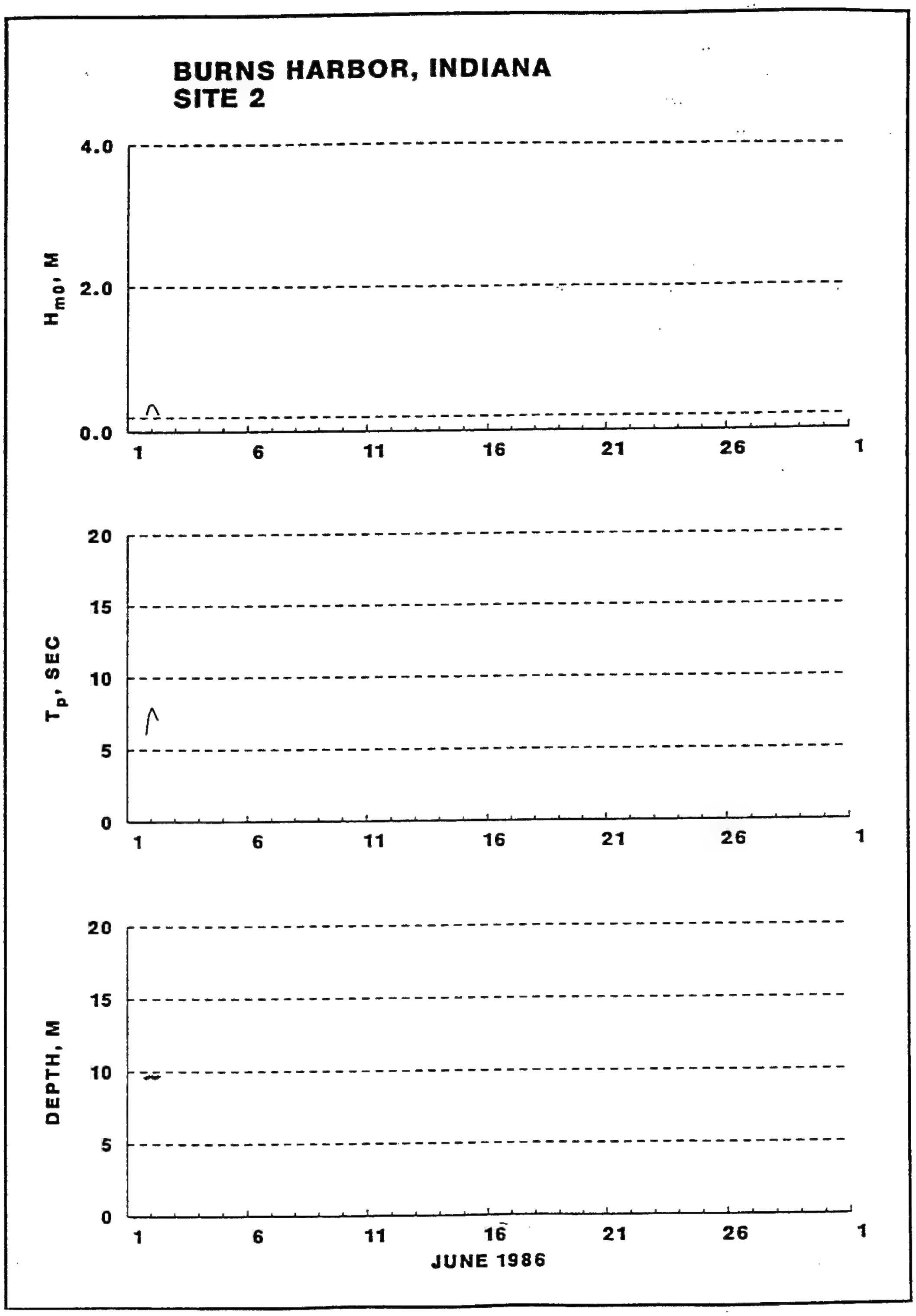
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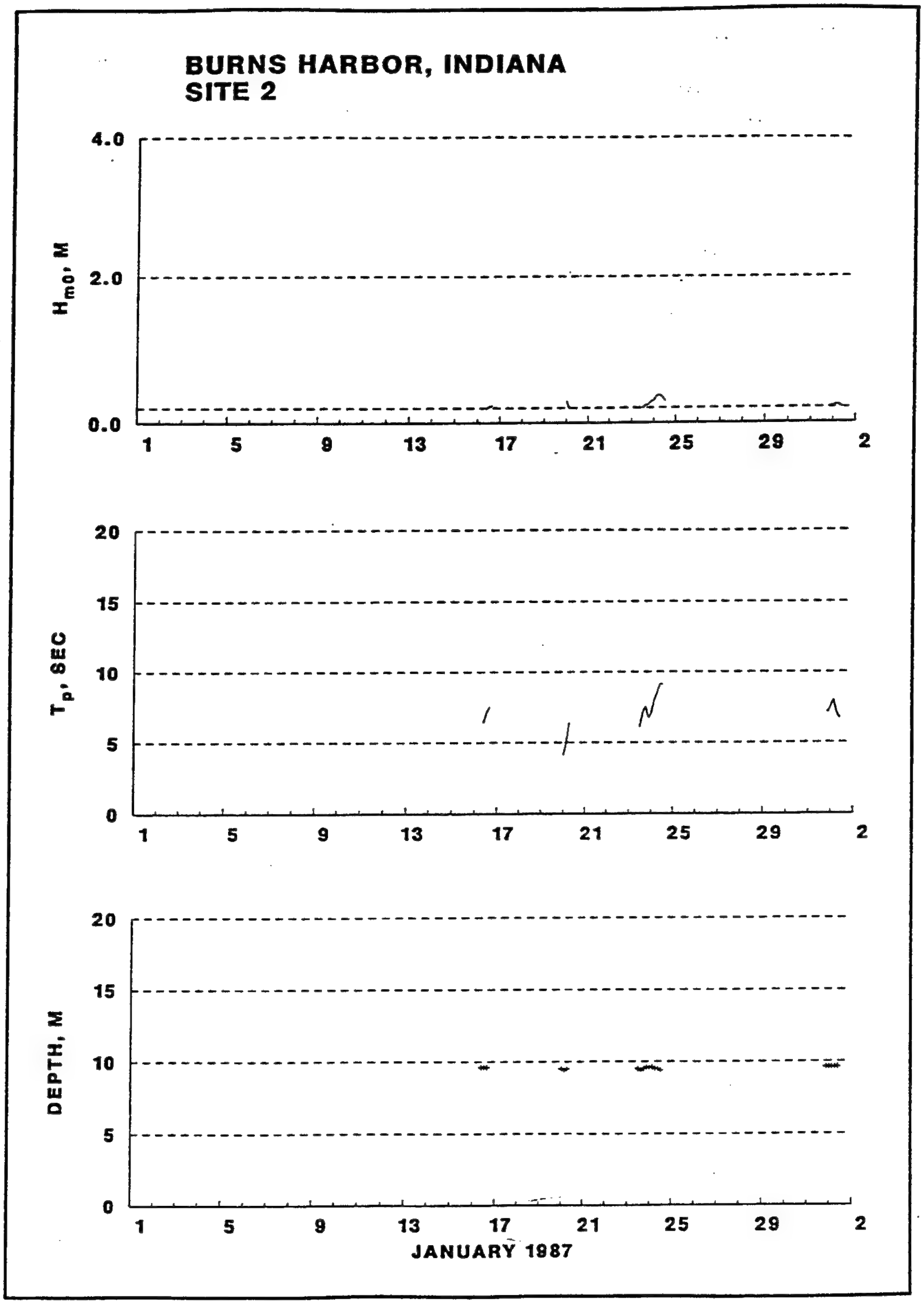

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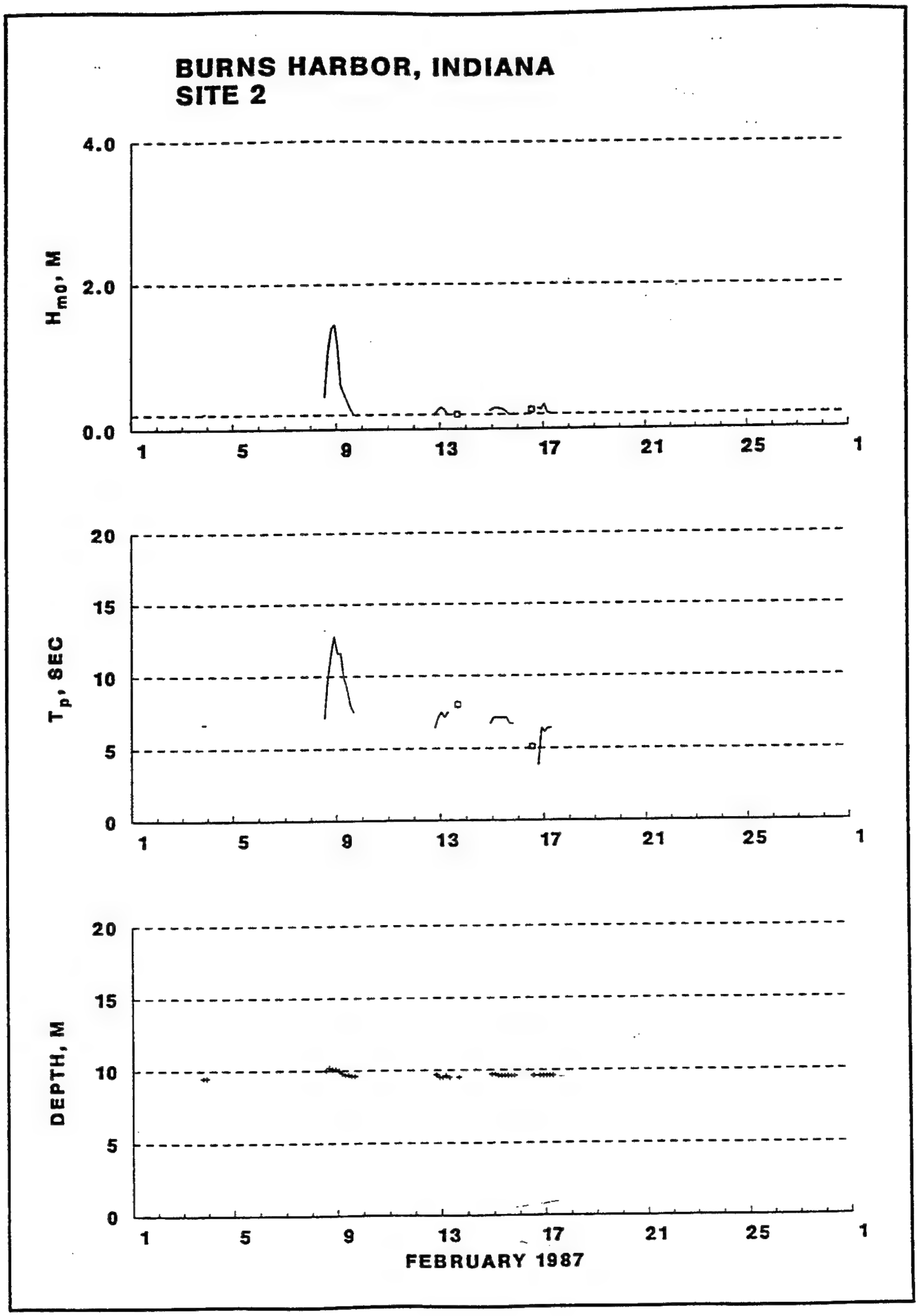
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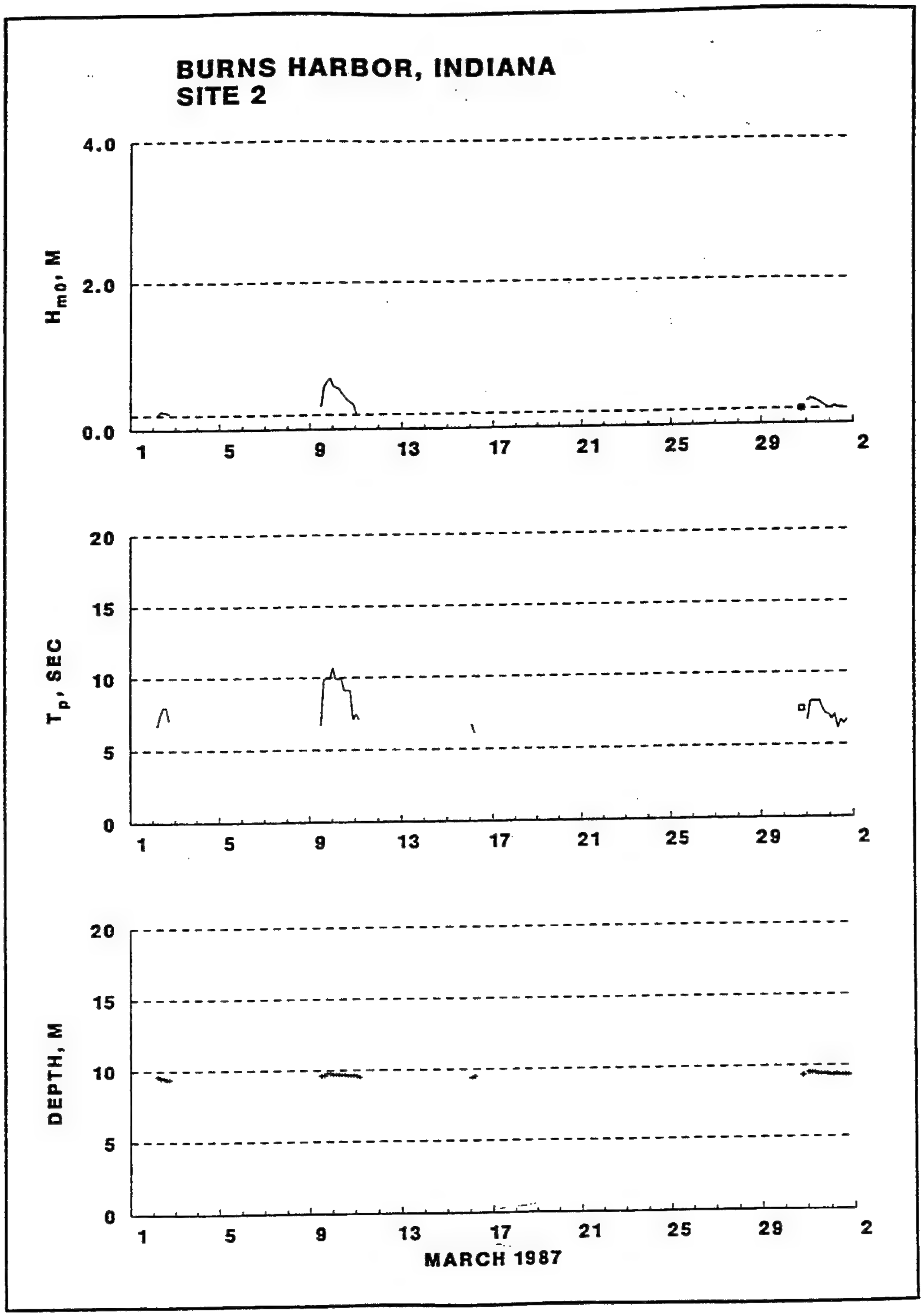
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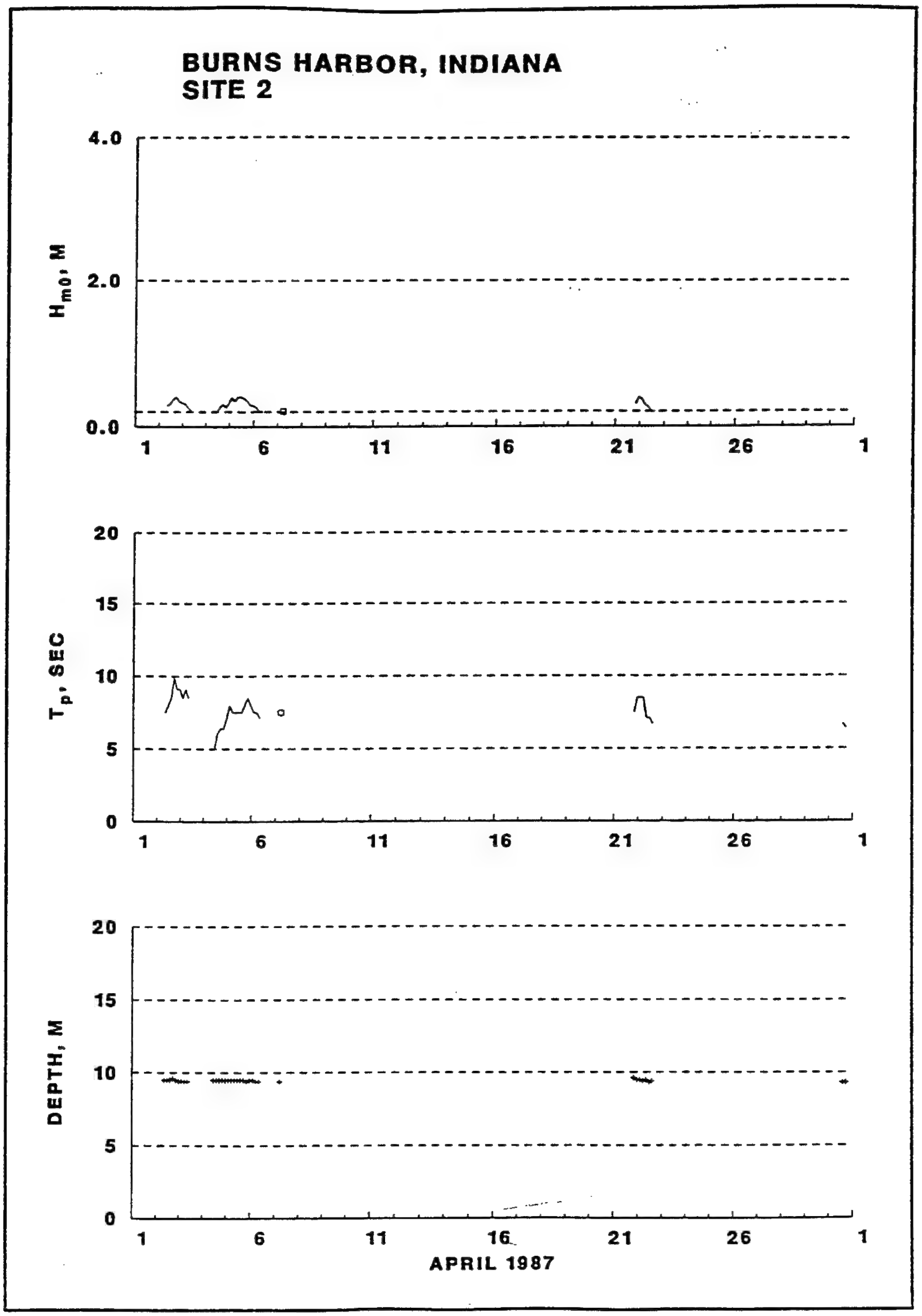

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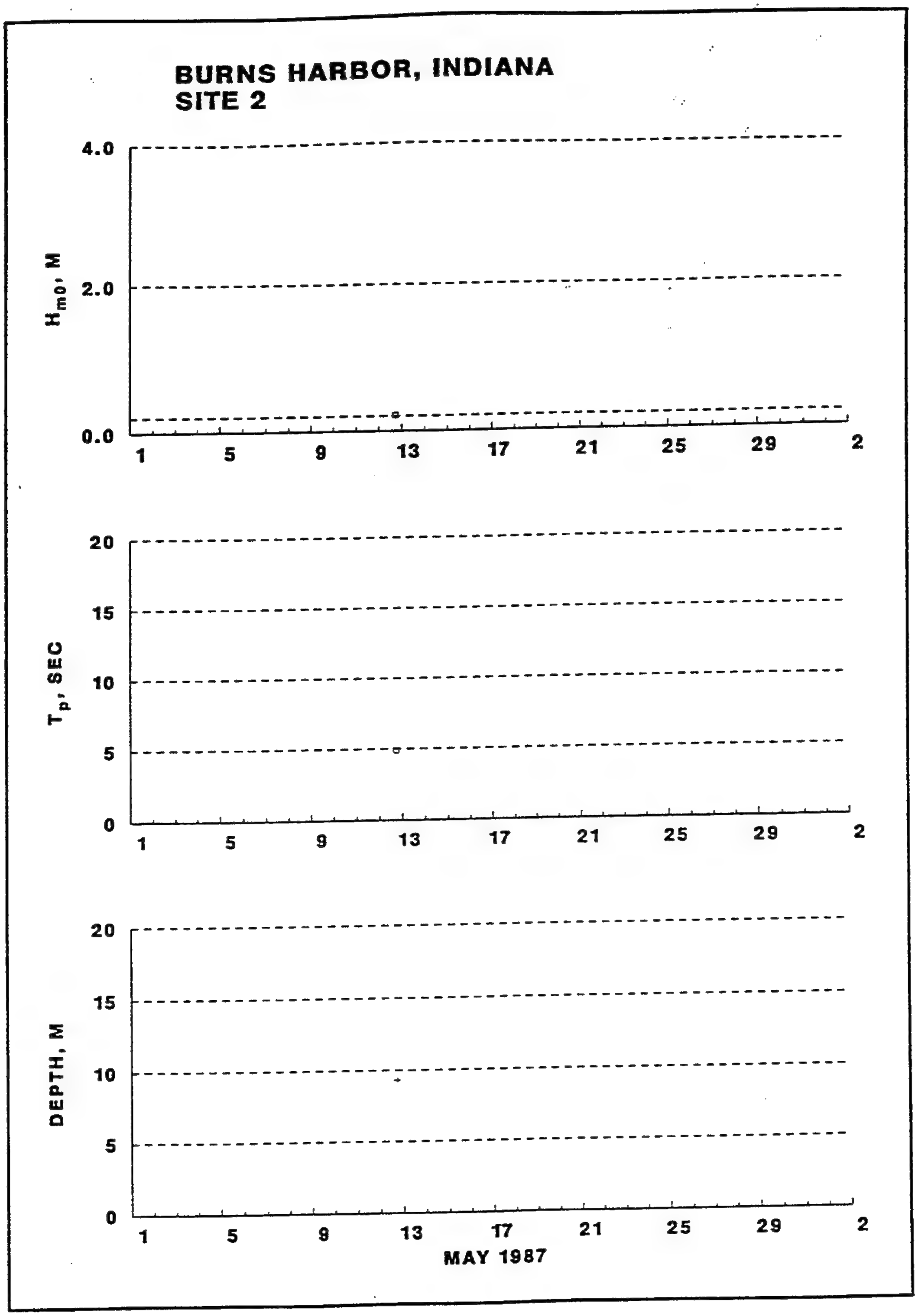
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MAY 1987

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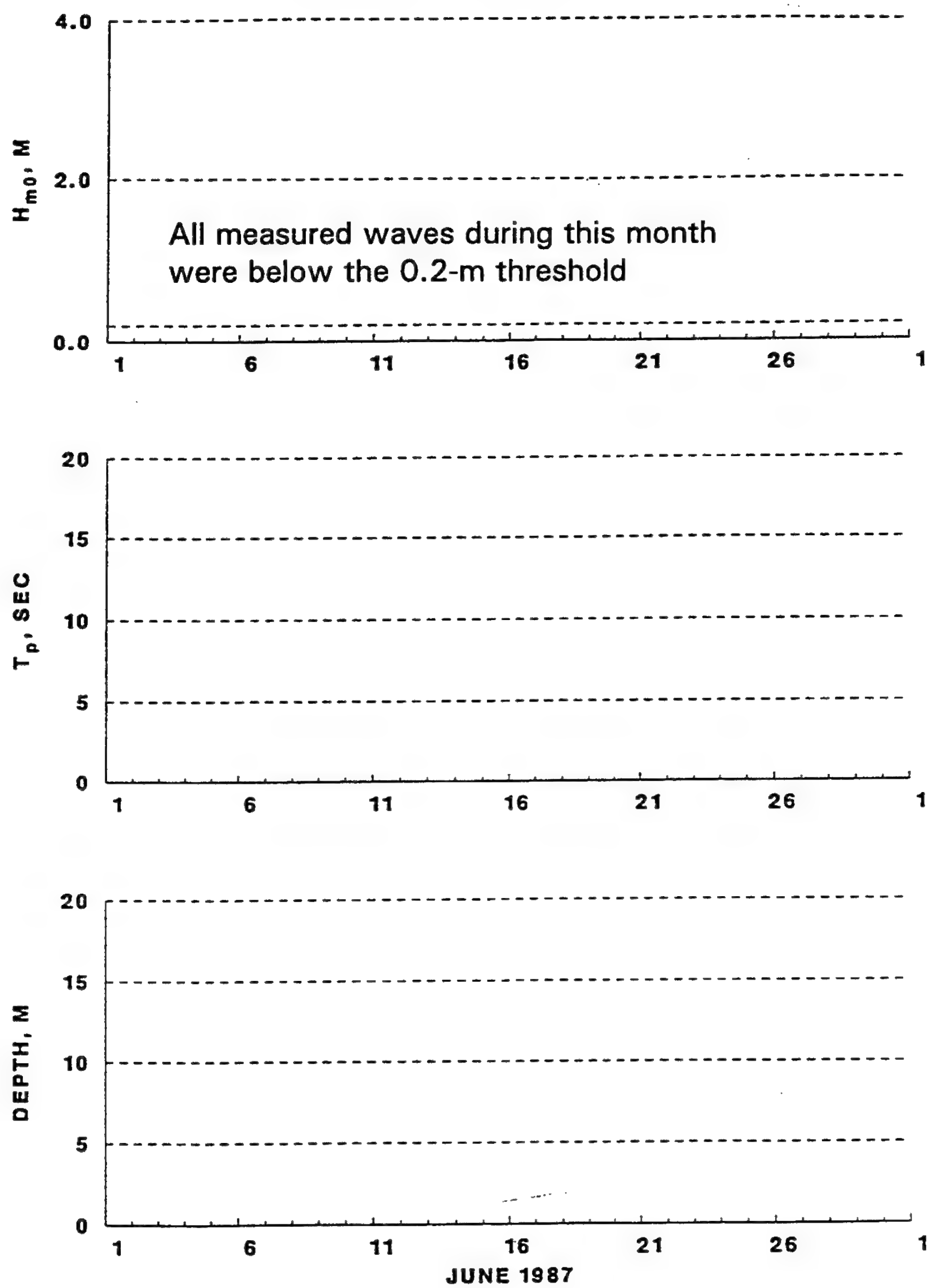

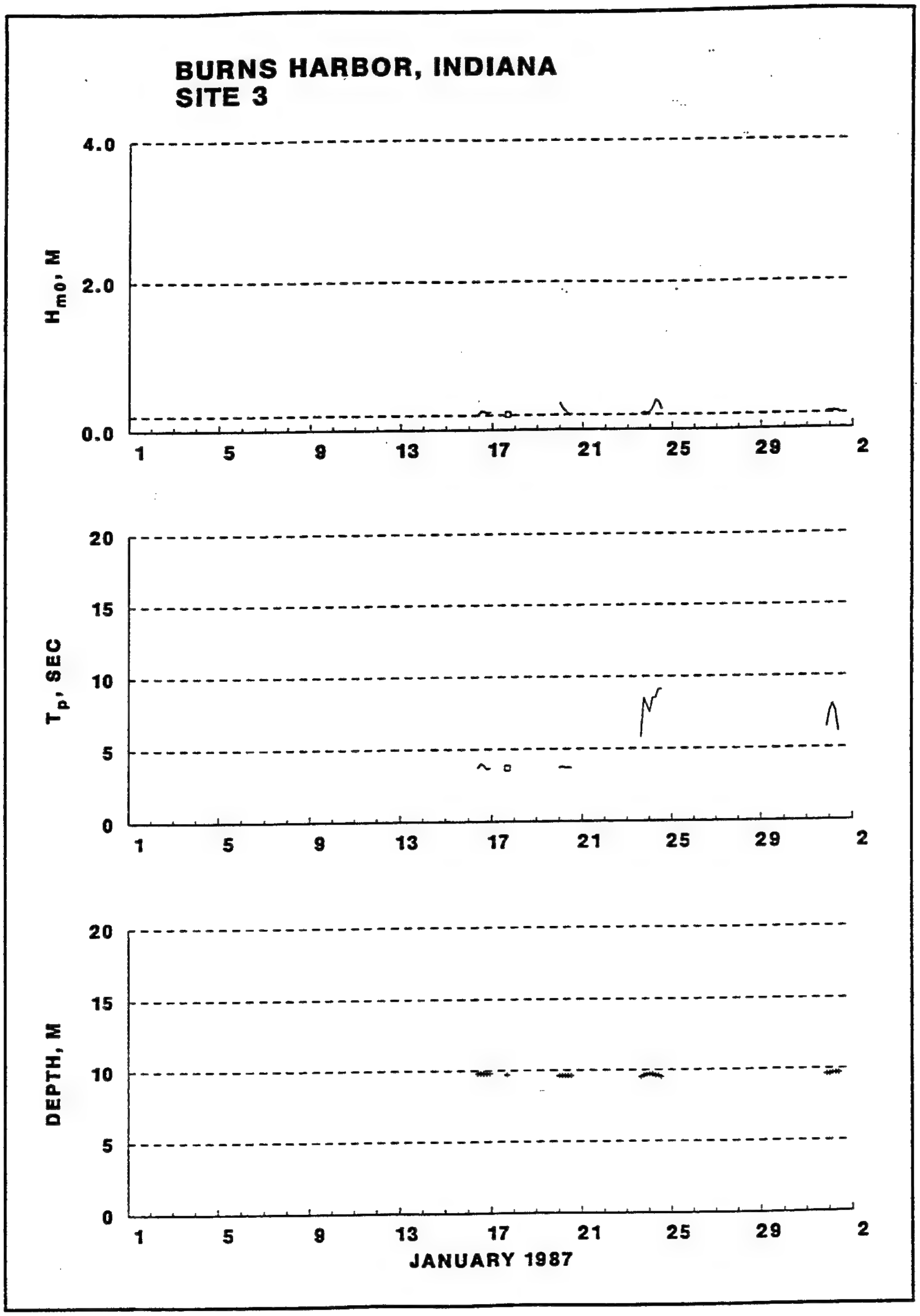
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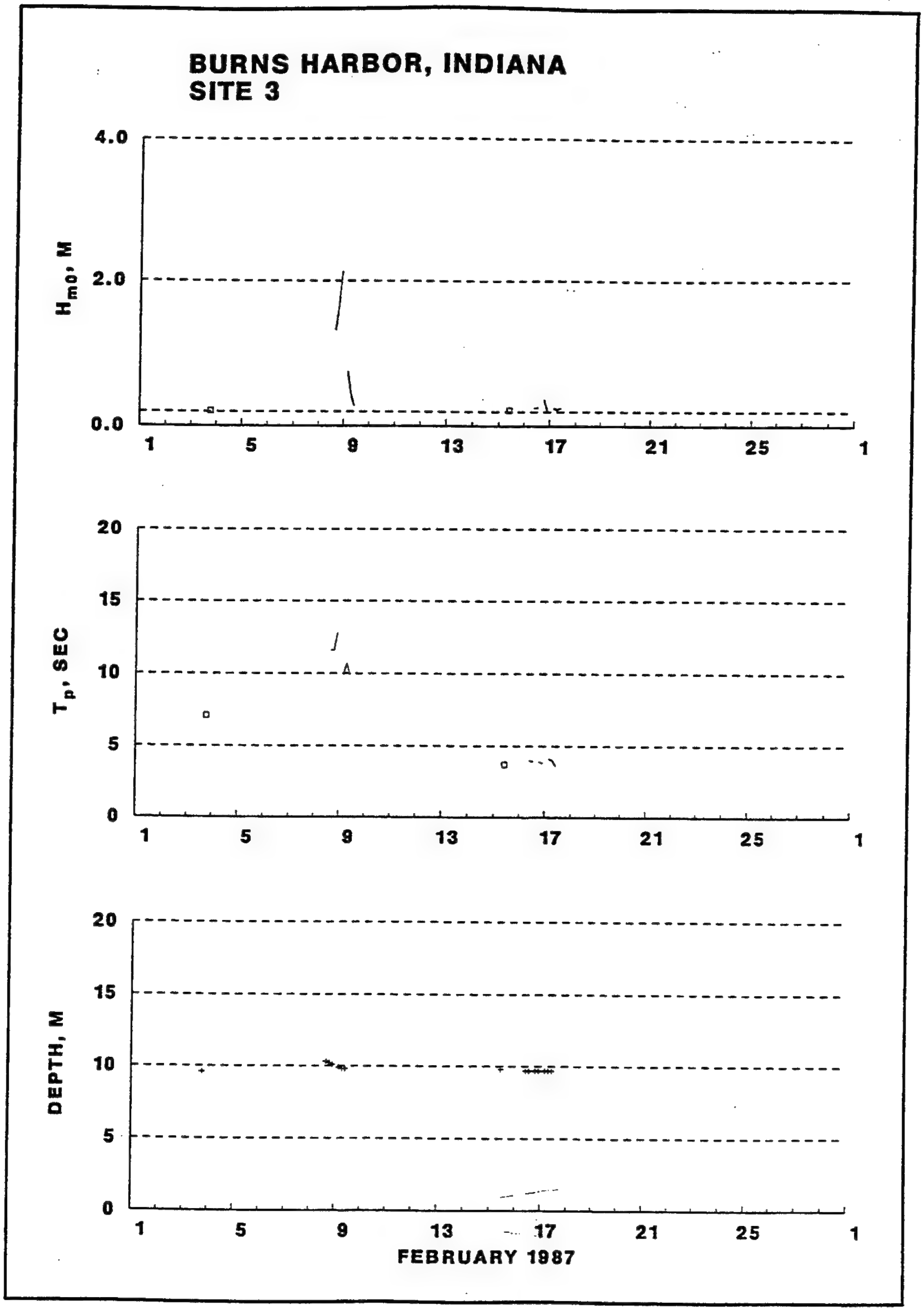
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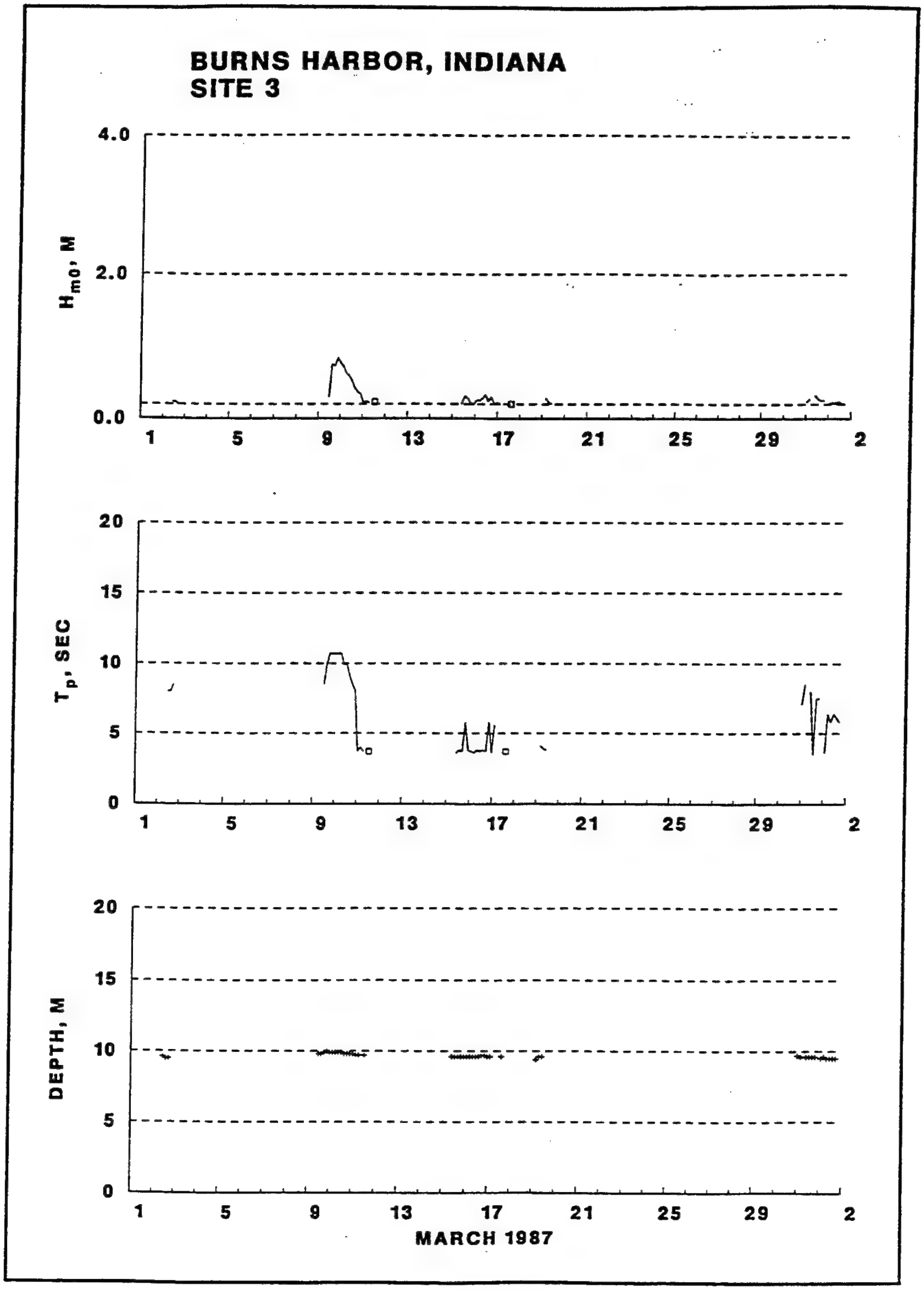
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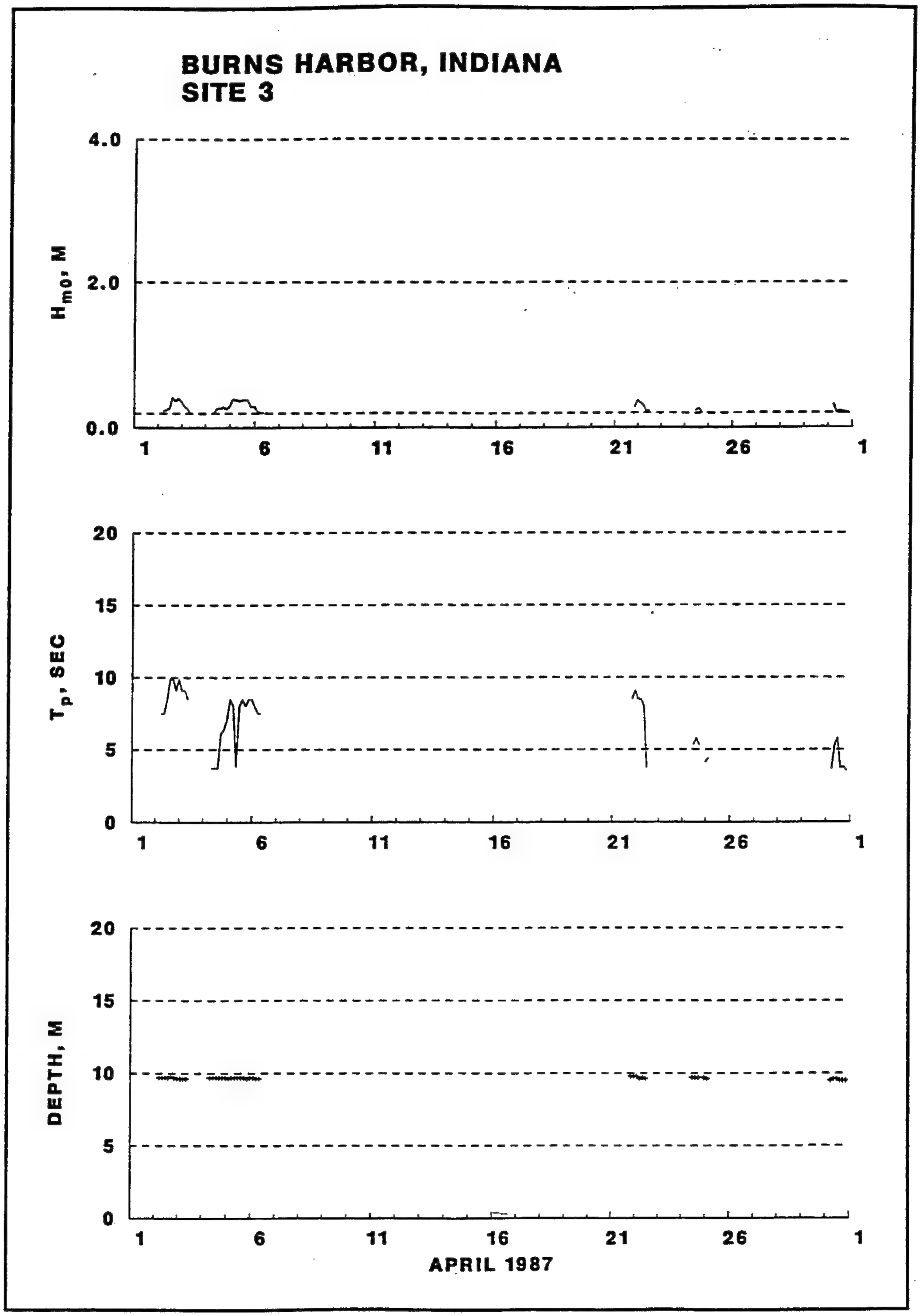
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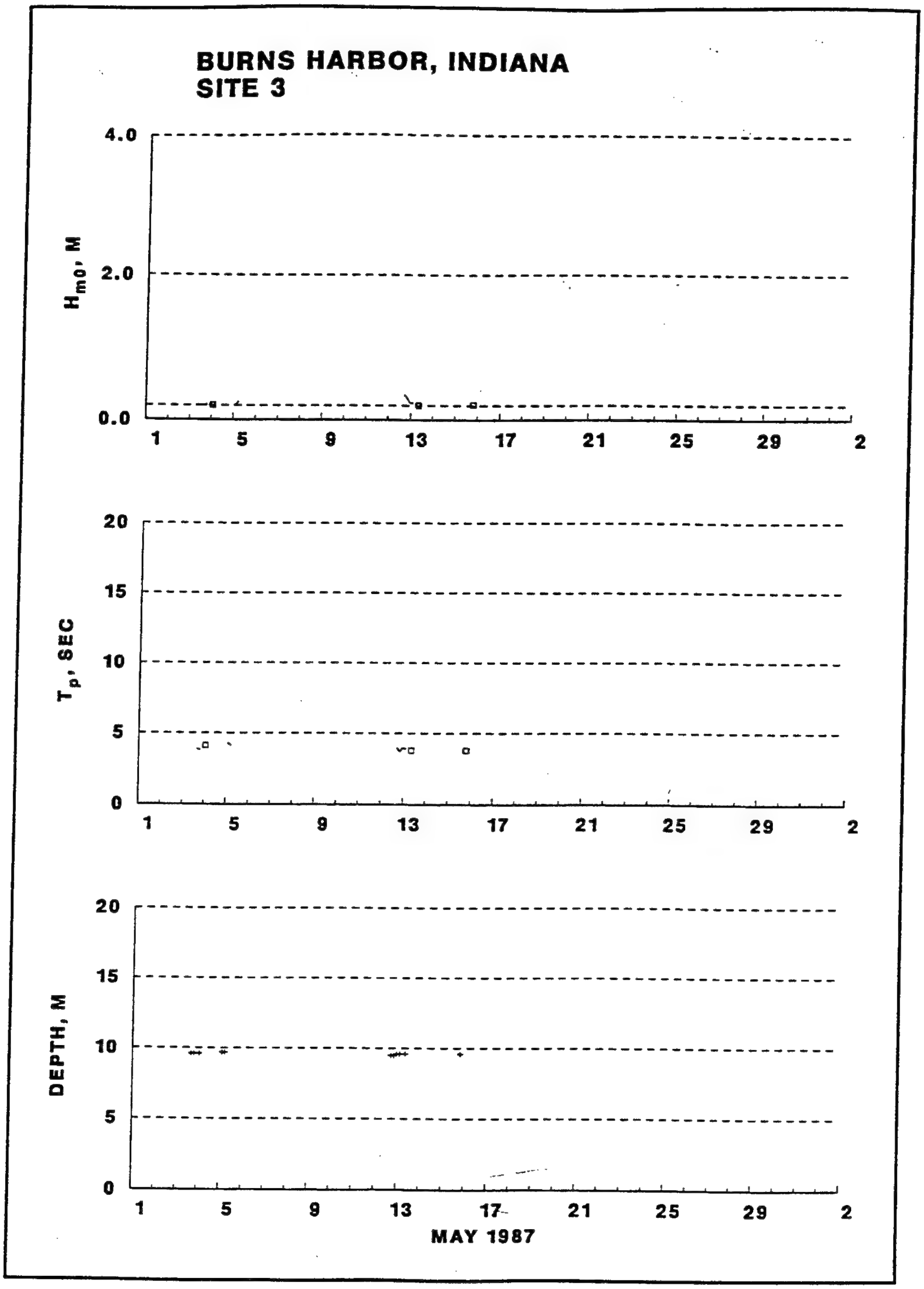
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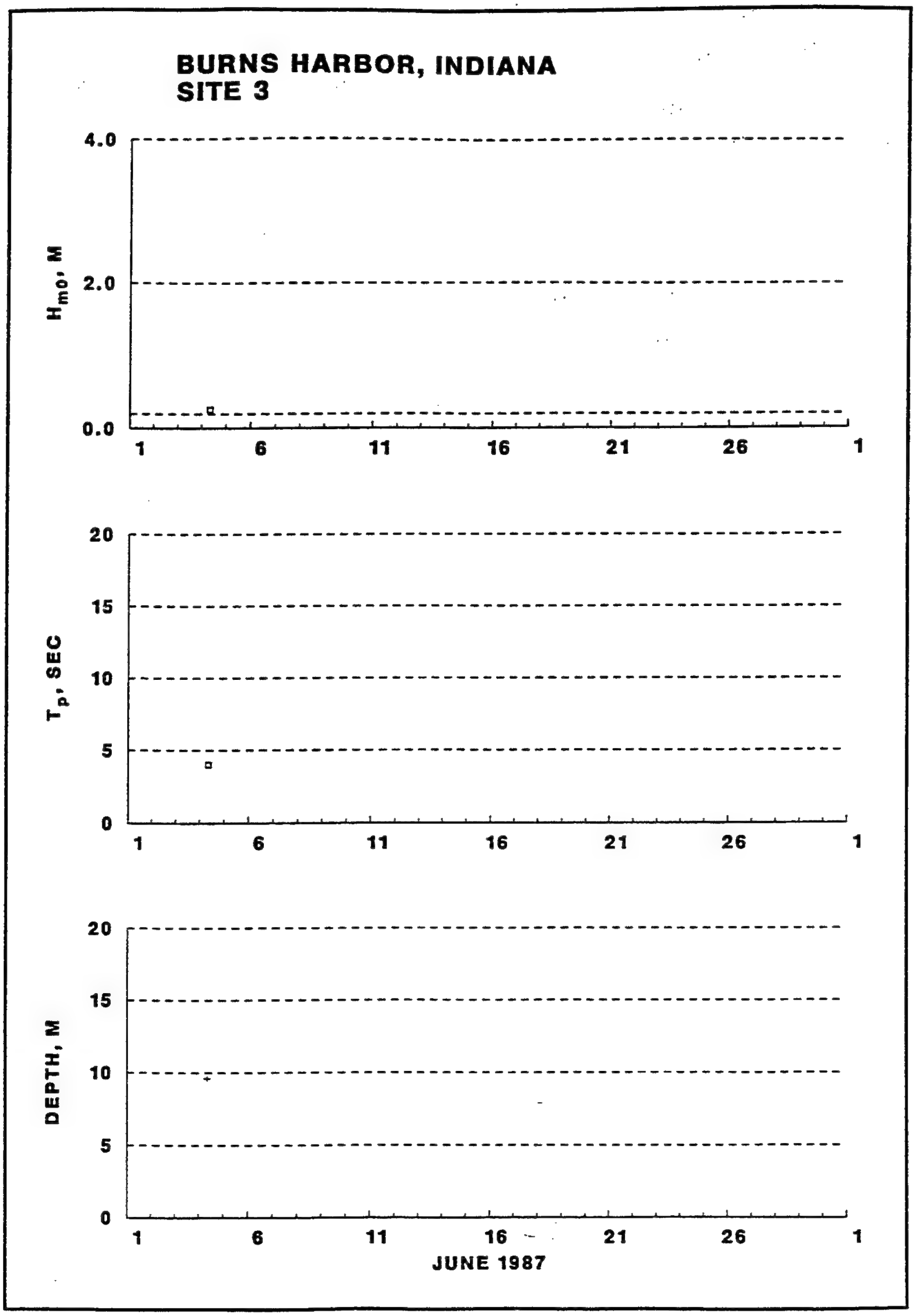
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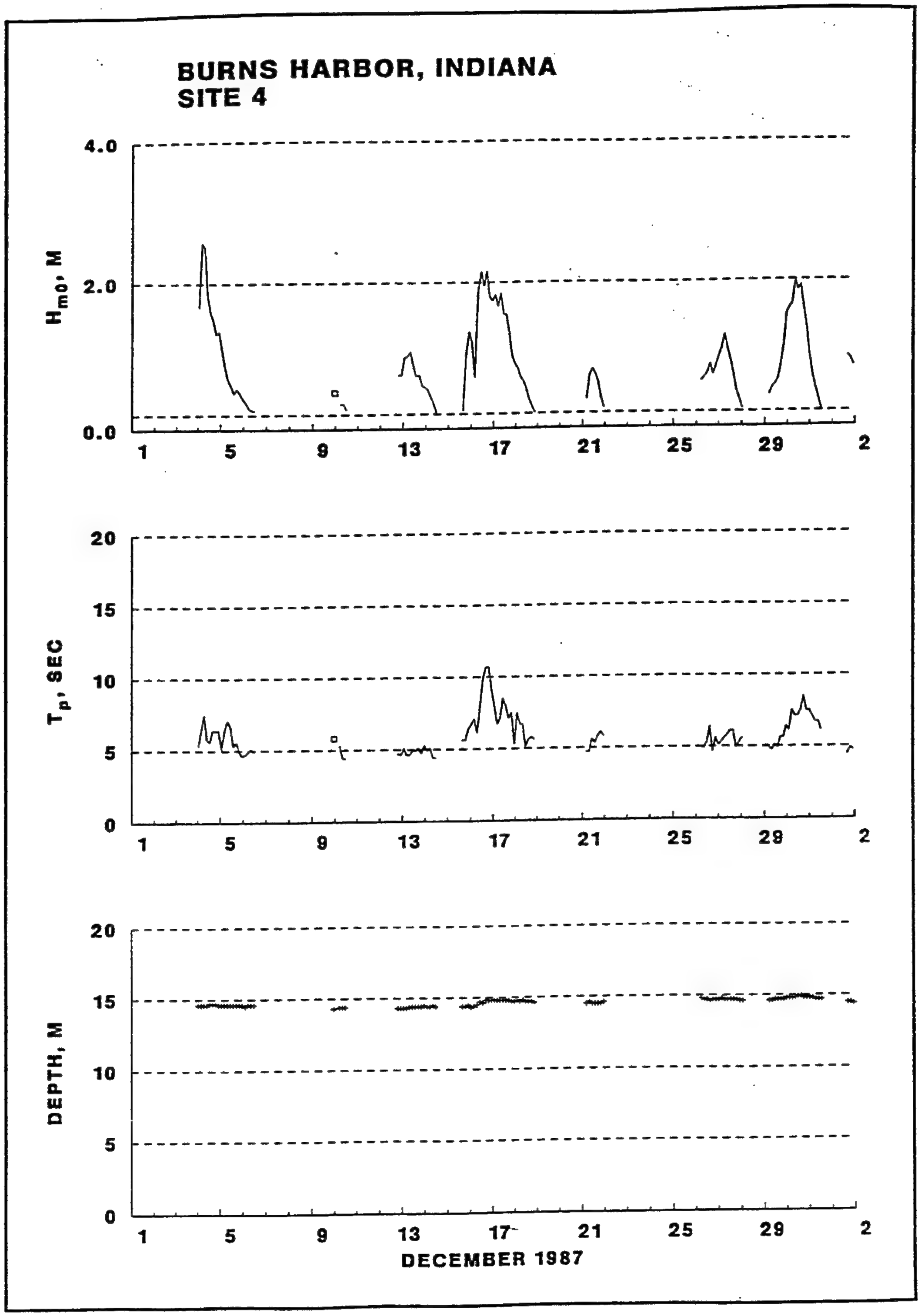

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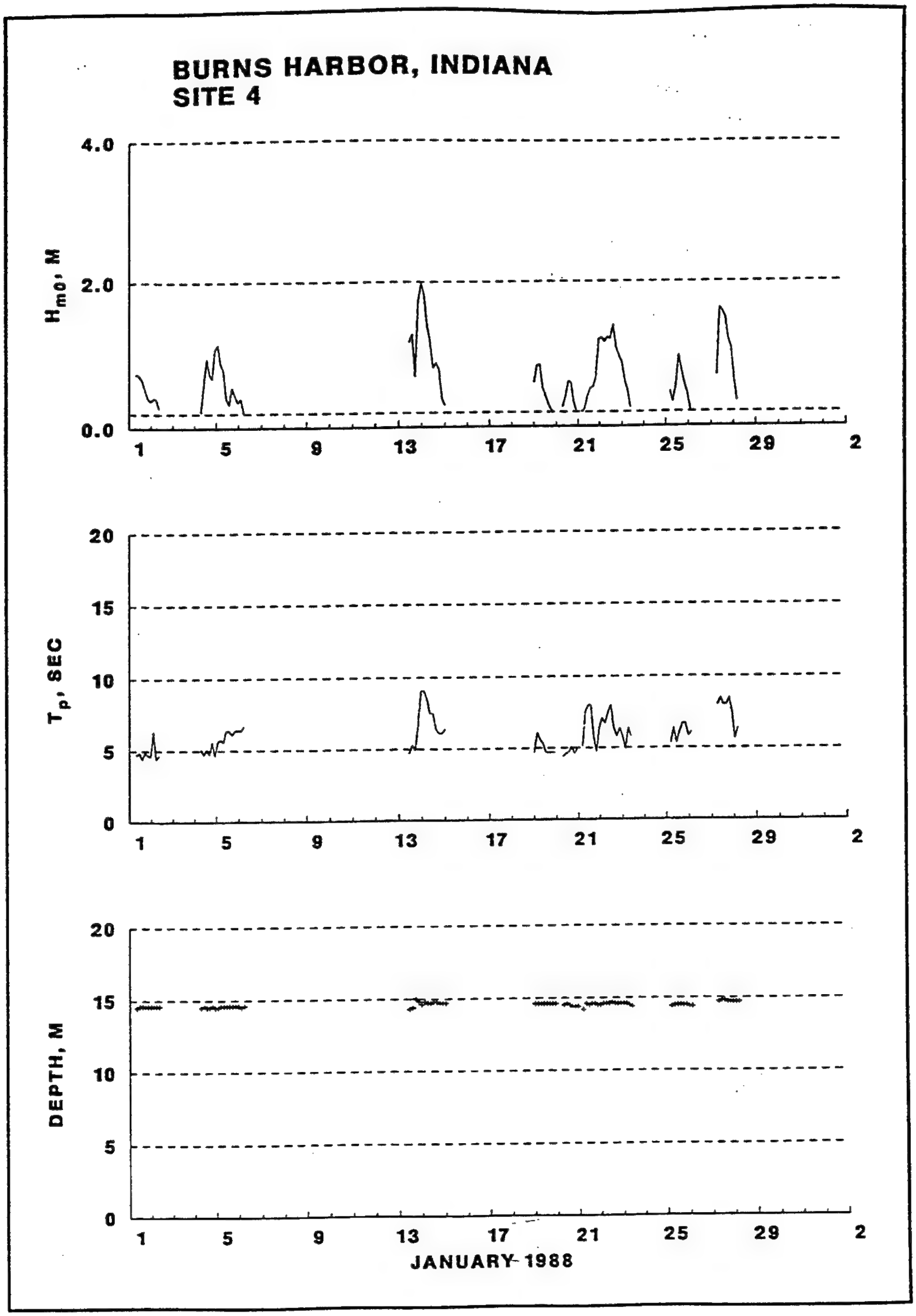
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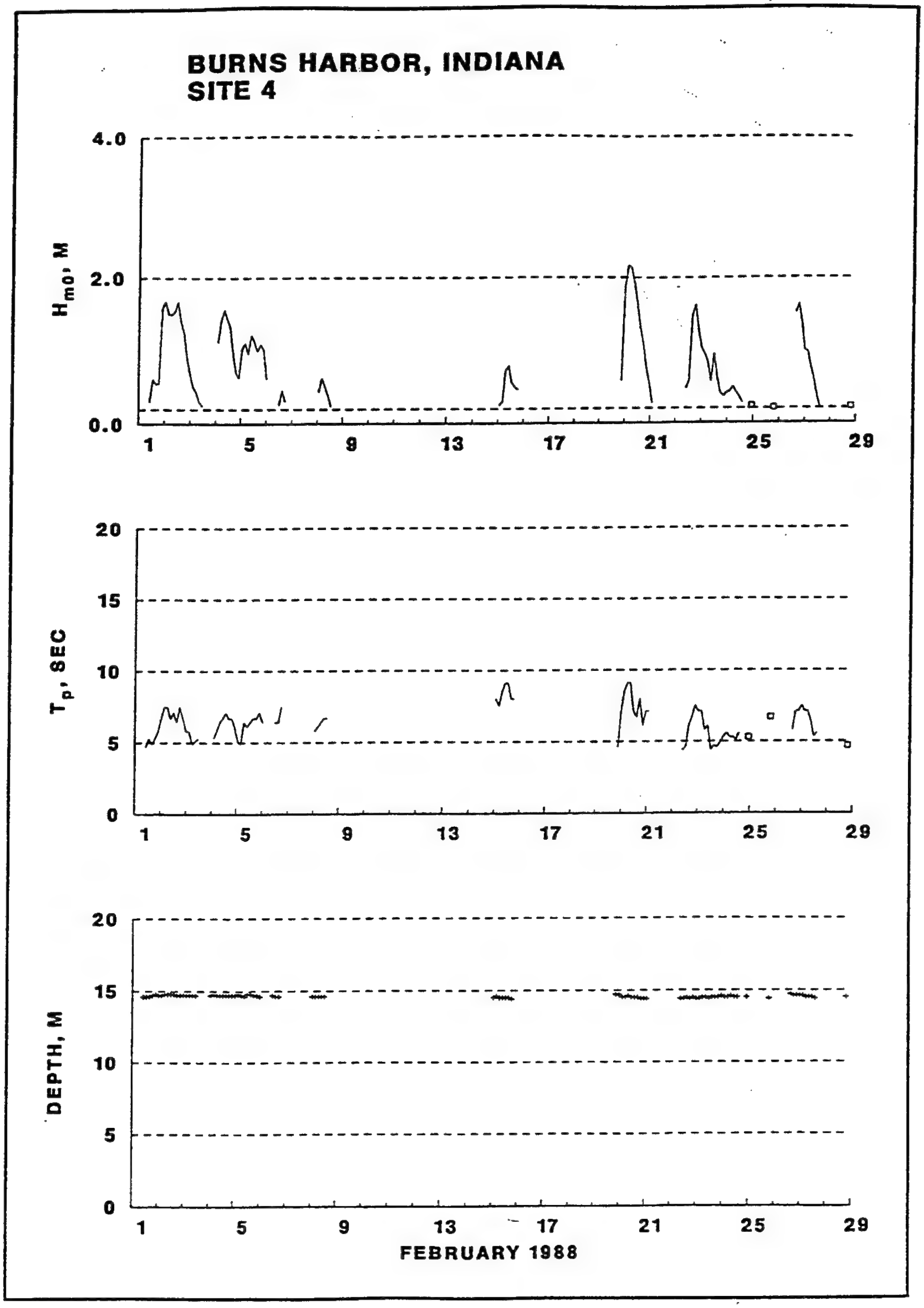

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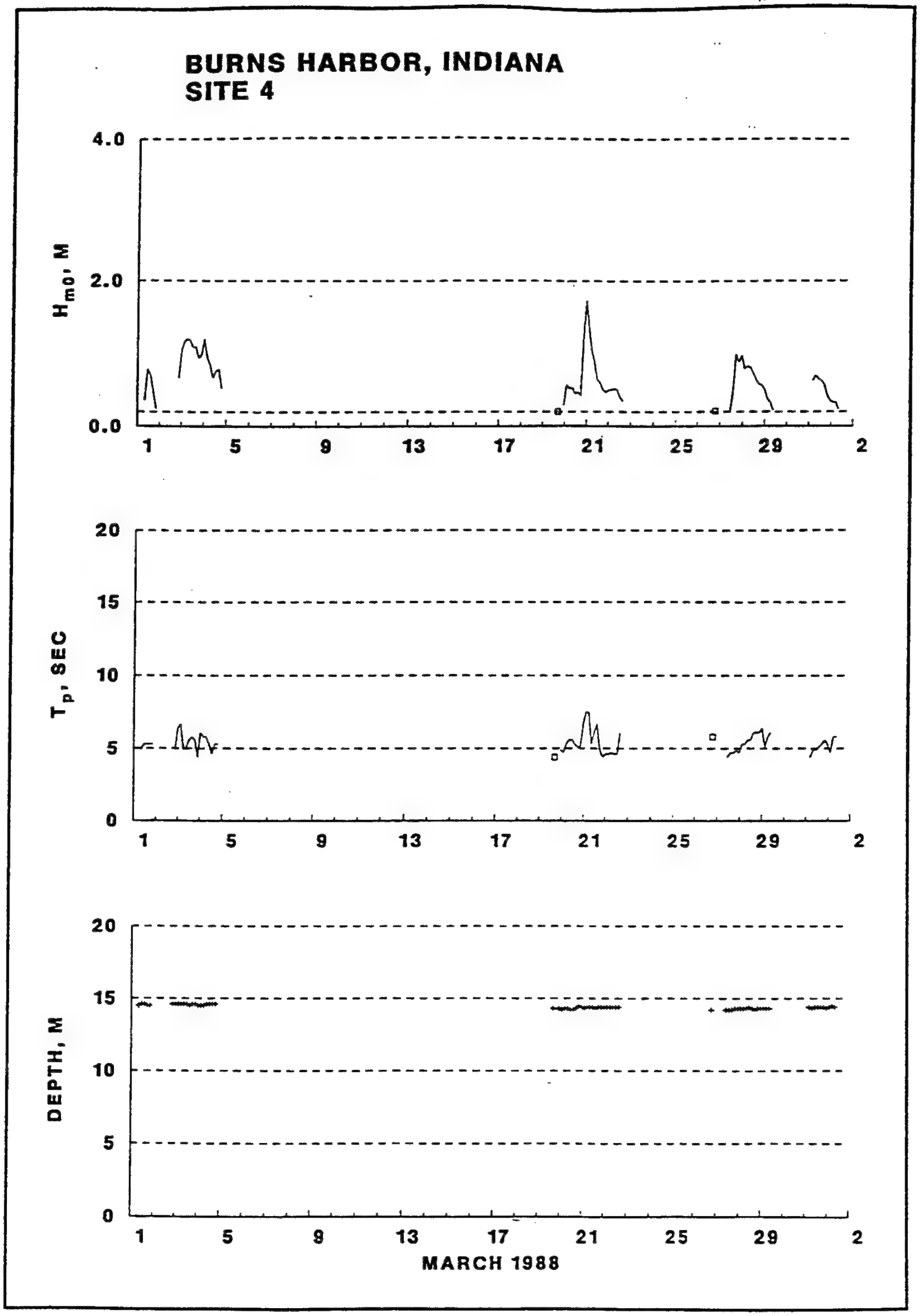
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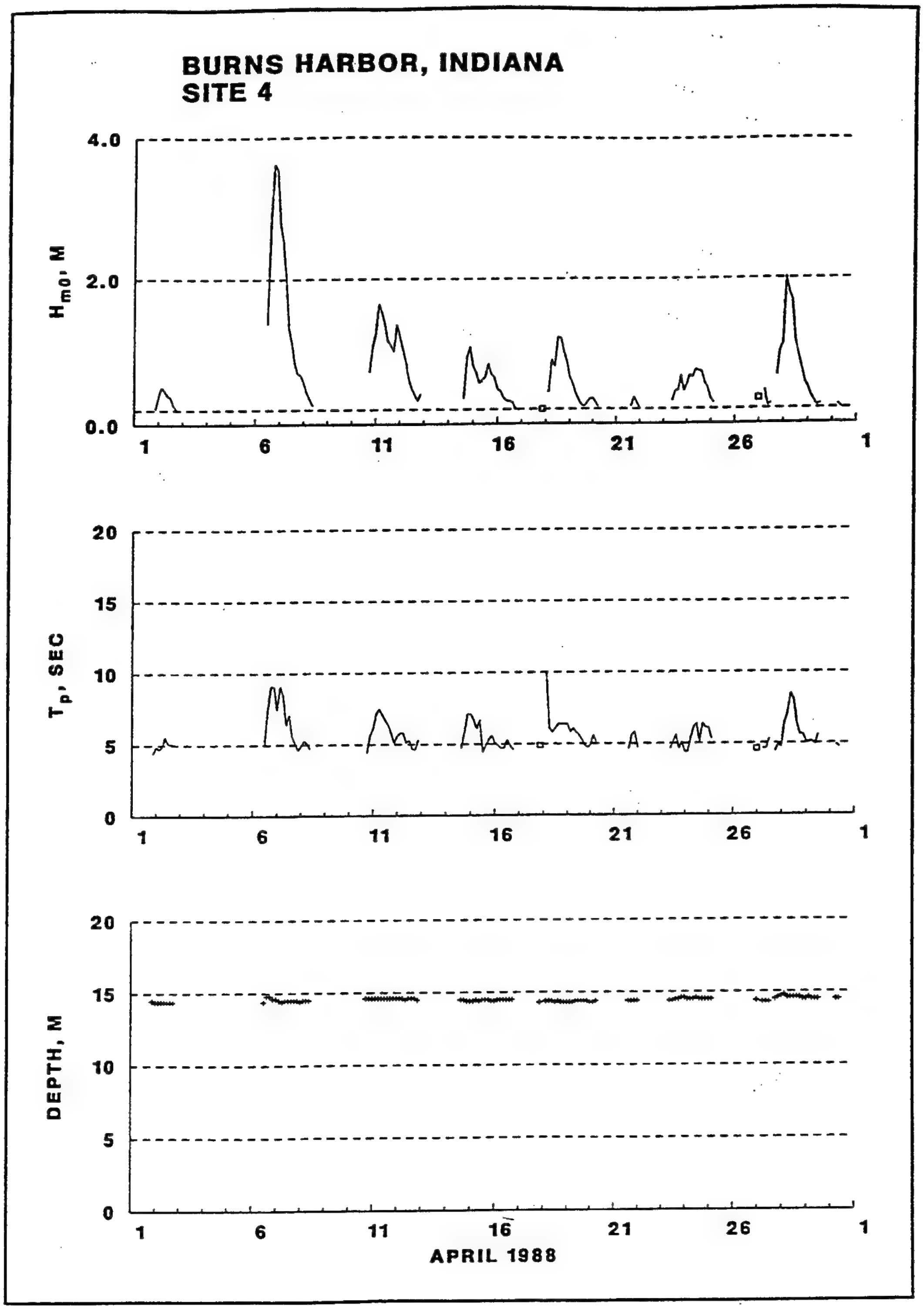
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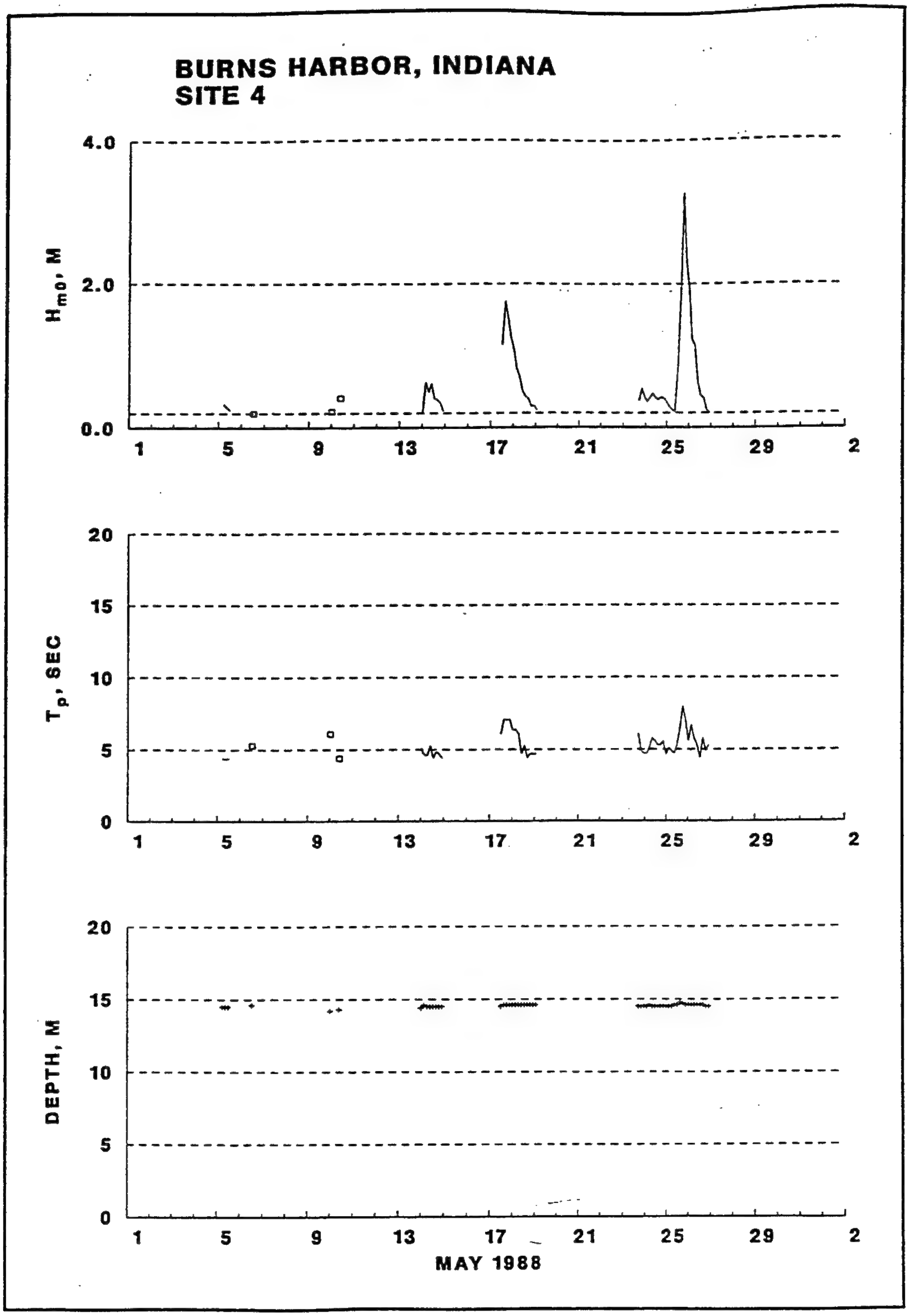

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MAY 1988

3 Extremal Analysis of Burns Harbor Hindcast and Measured Wave Data

by Michael E. Andrew, PhD[1]

Introduction

This study consists of a comparison of hindcast and measured wave data for Lake Michigan and in particular for the Burns Harbor area. This study was performed to provide verification of the WIS extremal analysis and to determine the effect, if any, of apparent disagreement between measured and hindcast data for the Burns Harbor area. This effort was initiated because of apparent inconsistencies between hindcast and measured data for the area of interest. The measured data sets consist of data taken from the National Data Buoy Center (NDBC) directional buoy 45007 located roughly 80 miles (129 km) north of Burns Harbor, and of pressure sensor data measured outside Burns Harbor that was corrected to yield only incident wave energy. Hindcast data were obtained from Wave Information Study (WIS) Station 62 near Burns Harbor and WIS Station 64 located near the NDBC buoy site.

It is necessary to perform an extremal analysis in any situation where design wave information is important. The difference between an extremal analysis (which provides actual return period waves and associated parameters such as nonencounter probabilities) and an ordinary statistical wave climate (which provides the frequency distribution of waves for a location) is that the extremal analysis deals only with storm or extreme event conditions. The extremal analysis is performed in such a way that only information pertaining to types of events that could be considered a threat to coastal structures are considered. The extremal analysis seeks to remove all wave heights that are not related to storms. Then, results of the extremal analysis can be applied to

[1] Consultant, Jackson, MS.

future storms that may threaten a coastal structure. A more detailed explanation of extremal analysis methods is found in Borgman and Resio 1982.

Wave Parameter Comparison for NDBC Buoy 45007 and WIS Hindcast Station 64

Both the WIS hindcast data and the NDBC buoy 45007 data (Appendix 3A) are for the period 6 April 1985 through 5 November 1987. All data provided by NDBC were such that the measured significant height was 6.6 ft (2 m) or more. The results of this analysis are for use in producing an extremal analysis so that the smaller waves are considered extraneous. To make comparisons, the two data records were combined and matched by time. Buoy data were subsampled by selecting the hourly value that coincides with the WIS time value. Scatterplots of the various parameters were used to make initial judgements about possible relations and agreement between the parameters from the two data records. Figure 3-1 displays the relationship between significant wave heights from the WIS hindcast and the NDBC measurements. It is apparent from Figure 3-1 that there is almost no relationship between the two wave height records. The squared correlation between the significant heights was $r^2 = 0.04$, indicating that the two significant height records explain only about 4 percent of the variability in each other. The squared correlation between two variables is defined to be the proportion of variability in one variable that is explained by the other variable, and is computed with the formula:

$$r^2 = \left[\frac{\Sigma xy - \dfrac{(\Sigma x)(\Sigma y)}{n}}{\sqrt{\left[\Sigma x^2 - \dfrac{(\Sigma x)^2}{n}\right]\left[\Sigma y^2 - \dfrac{(\Sigma y)^2}{n}\right]}} \right]^2 \tag{3-1}$$

Figure 3-2 displays a plot of the difference between the NDBC and WIS significant height (WDIFF = NDBC - WIS) plotted against the difference in associated peak periods (PDIFF = NDBC - WIS). There is very good agreement between the two differences and it is not surprising that errors in significant height would be positively related to errors in peak period. The squared correlation between the wave differences (WDIFF) and period differences (PDIFF) was $r^2 = 0.71$. Figure 3-2 is of little help in deciding which data set to believe; however, persistent errors in significant height with accompanying errors in peak period are more likely to result from problems in a hindcast rather than problems with data collection and analysis methods. The wind speeds used in performing a hindcast are a likely cause for persistent error in an accurate hindcast model. Errors in wind speeds could produce the correlated errors of Figure 3-2, since both height and period depend heavily on wind speed and duration.

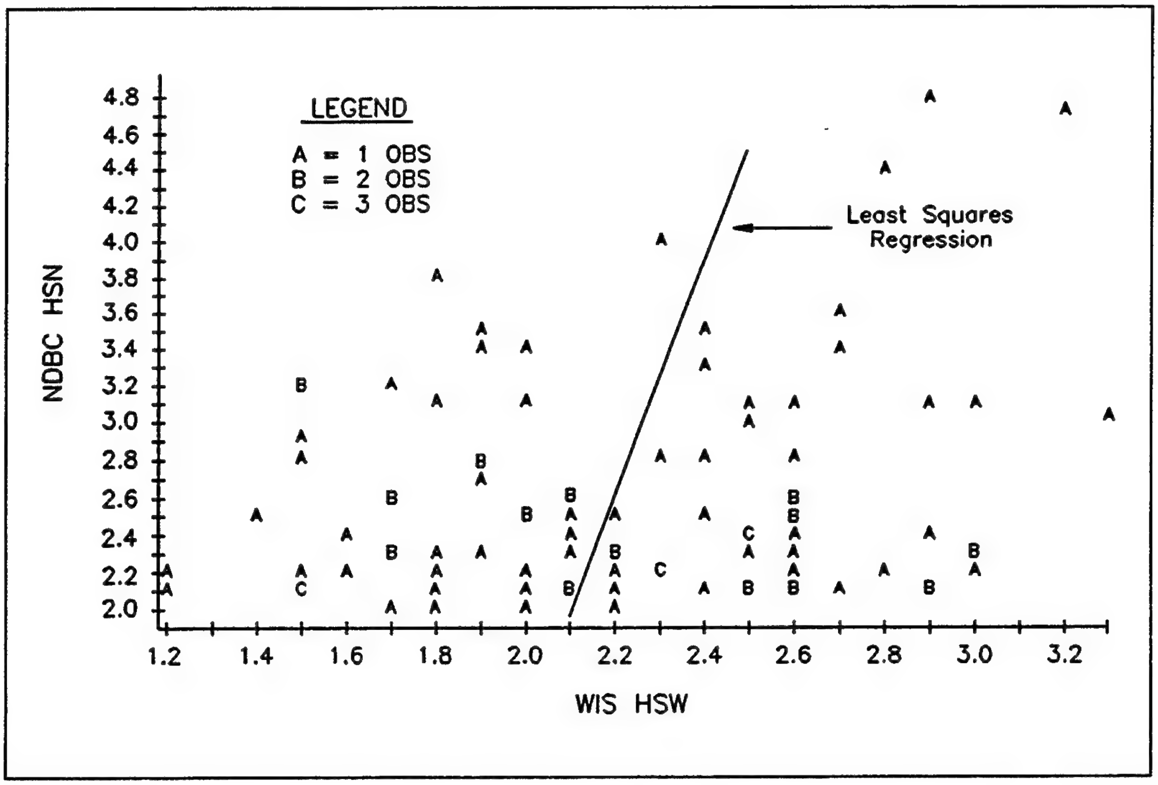

Figure 3-1. Relationship of wave heights from both WIS hindcast and NDBC data

Figure 3-3 represents the difference in significant heights (WDIFF) versus the differences in wind speeds between the NDBC data and WIS data (SDIFF = NDBC - WIS). As expected, there is a clear linear relationship between disagreement in wind speed and disagreement in significant height for the two sets of records.

Figure 3-4 demonstrates a similar relationship between disagreement in wind speed (SDIFF) and disagreement in peak period (PDIFF). However, the relationship is weaker than for significant height (reflecting the less direct relationship between wind speed and peak period). Since disagreement in peak periods is the best single predictor of disagreements in significant height, and disagreement in wind speed has a relatively weak relationship with disagreement in peak periods, a multiple regression was performed to see how disagreement in peak period (PDIFF) and wind speed (SDIFF) predict disagreement in significant height (WDIFF). The overall squared correlation was $r^2 = 0.83$, meaning that PDIFF and SDIFF explain 83 percent of the variability in significant height errors between the hindcast and measured data records. The additional correlation analyses were performed to assist in identifying potential sources of problems in the hindcast.

To determine the validity of the claim that input parameters for the hindcast are the most likely cause of disagreement in this data set, it is necessary to obtain other wind speed information for the nearby area.

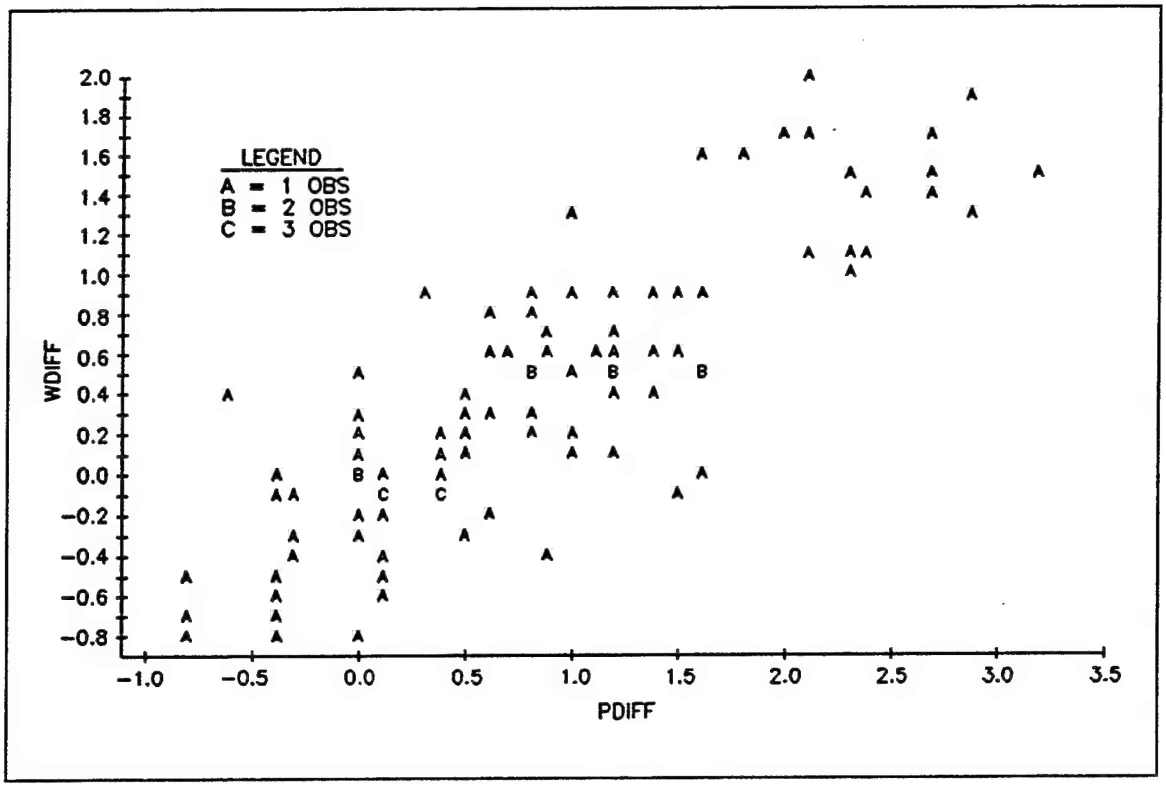

Figure 3-2. Difference between NDBC and WIS significant height (WDIFF = NDBC - WIS) plotted against the difference in associated peak periods (PDIFF = NDBC - WIS)

Limited comparisons were made using wind speeds taken from NDBC buoy 45002, located on the north side of Lake Michigan. Table 3-1 contains values for wind speed from both NDBC buoys and the WIS hindcast along with wave parameters for NDBC buoy 45007 and the nearby WIS hindcast (Station 64).

Table 3-1 represents one of the several storm events for which disagreement between the hindcast and measured data are greatest. Most of these instances occur during 1987. Prior to 1987, there is much better agreement between significant heights and peak periods for the two data records. The measured wind speeds, significant heights, and peak periods (during storm events) in the 1987 data are nearly all greater than the hindcast values (see the data listing in Appendix 3A for more detail). Also note that where there are large differences between WIS and NDBC significant heights during 1985, there are also notable differences in wind speed. Looking at the column entitled WDIFF (NDBC-WIS sig. ht.) in Appendix 3A, it appears that on the average, except for the 1987 data, there is no overall bias between significant heights. In fact, the average difference or bias (the mean of WDIFF) is 0.2 ft (0.07 m) (not significantly different from zero, the probability of a greater mean difference by chance being $p = 0.3$) if the 1987 data are excluded. If the 1987 data are included, then the mean difference in

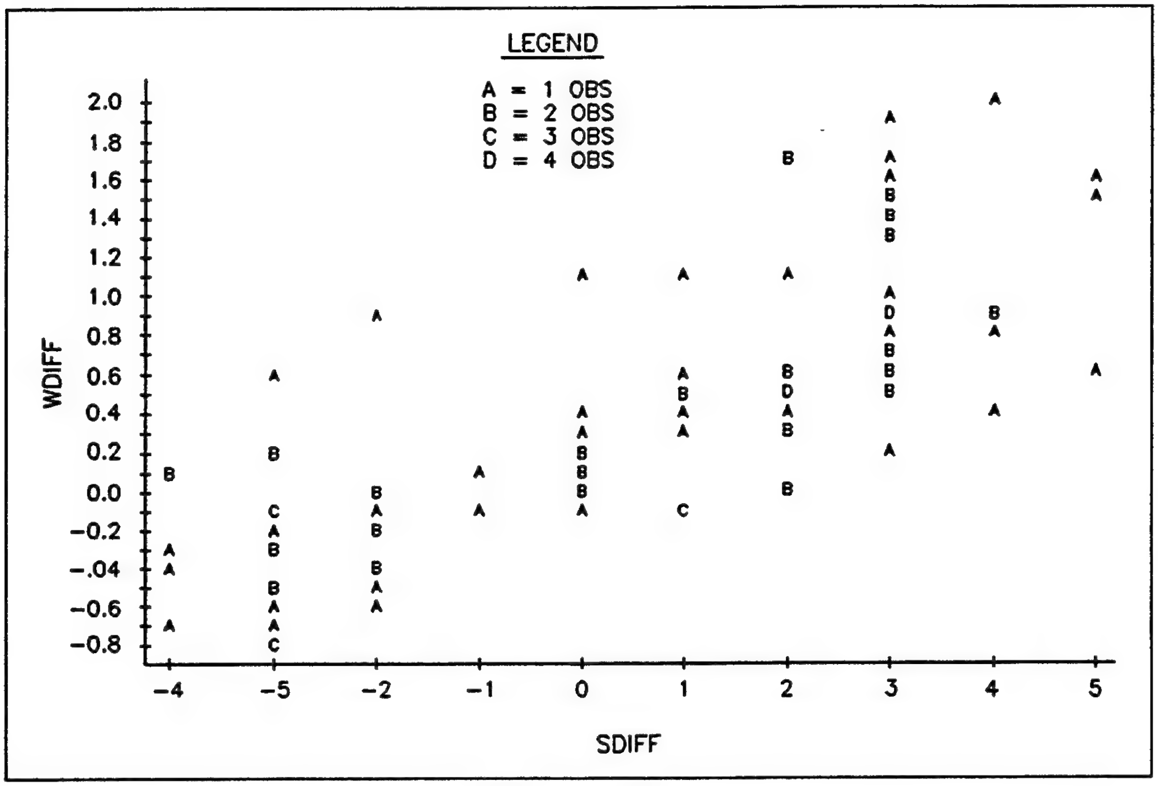

Figure 3-3. Difference in significant heights (WDIFF) versus difference in wind speeds between NDBC data and WIS data (SDIFF = NDBC - WIS)

significant heights between the WIS and NDBC data is 1.3 ft (0.4 m) (highly significantly different from zero, $p < 0.01$). A good measure of the hindcast data quality for extremal prediction is the long-term bias rather than correlation of instantaneous wave parameters. Then, for purposes of extremal analysis, exclusion of the 1987 WIS data and/ or replacement of the 1987 WIS data with the NDBC data is justified. WIS hindcast data prior to 1985 have been validated by WIS using the same NDBC buoy data. Its use in the following extremal analyses is justified by citing the WIS reports concerning this validation.

Wave Parameter Comparison for Burns Harbor Pressure Sensor Data and WIS Hindcast Station 62

Comparison between Burns Harbor pressure sensor data (corrected to yield only incident wave energy) and WIS Station 62 results in a somewhat stronger empirical relation than that of the previous section. The least squares regression between significant height for the two data sets results in a squared correlation of $r^2 = 0.31$, meaning the WIS hindcast explains about 31 percent

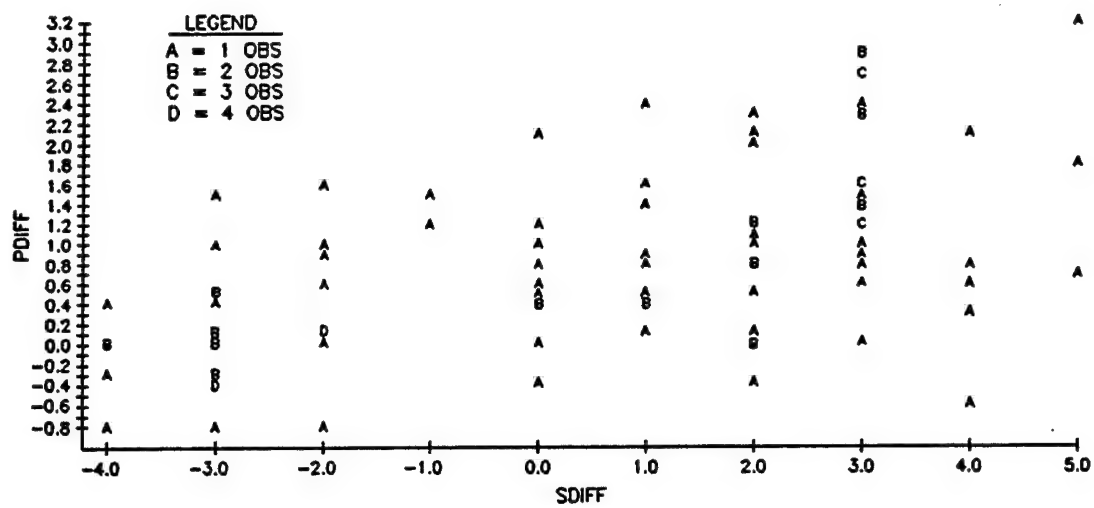

Figure 3-4. Relationship between disagreement in wind speed (SDIFF) and disagreement in peak period (PDIFF)

Table 3-1
Wind Speed Comparison

	NDBC Buoy			WIS	
	45007		45002		
Date	Hs m	Speed m/sec	Speed m/sec	Hs m	Speed m/sec
10/1/87	2.6	14	13	1.7	11
10/3/87	4.8	15	14	2.9	12

of the variability in the measured data. Figure 3-5 demonstrates the least squares regression along with a scatterplot of the data. Other wave parameters such as peak period were not helpful in improving the empirical relationship because of the limited overlap between data sets, which results in a relatively small sample.

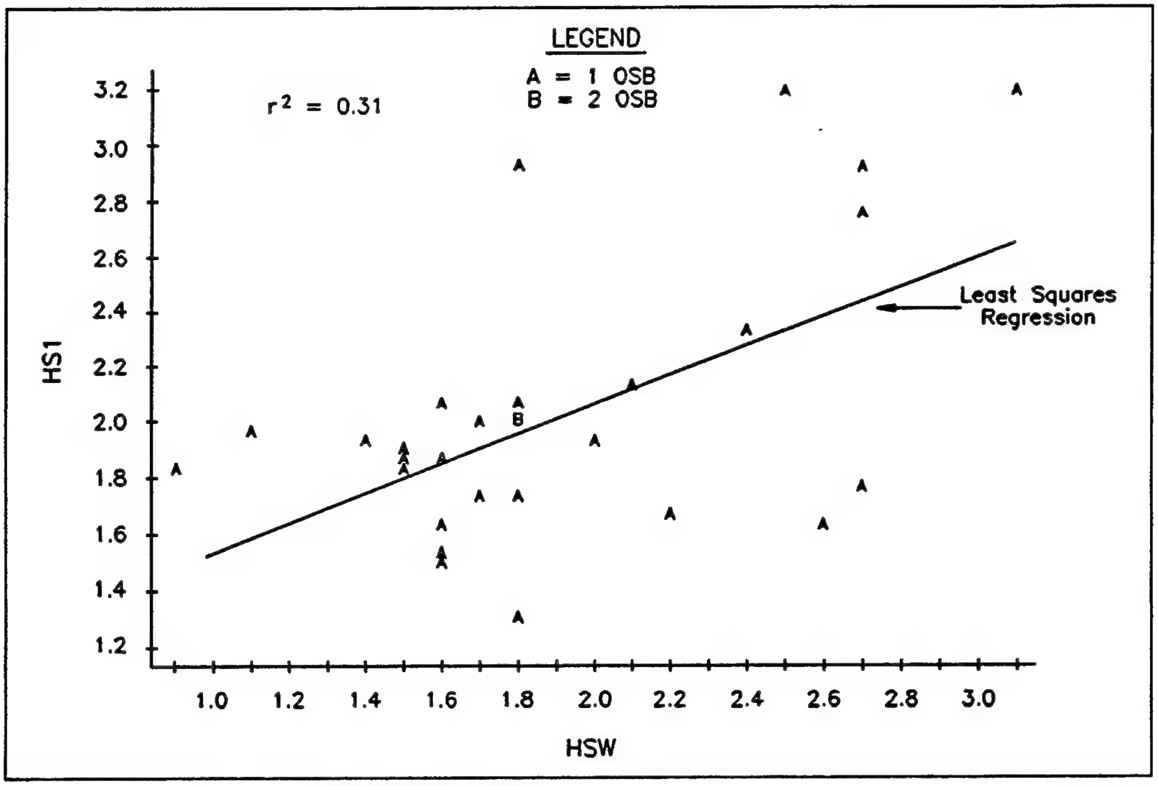

Figure 3-5. Plot of the least squares regression along with a scatter plot of the data

Extremal Analysis

Since strong empirical relations between measured and hindcast data are not available because of limited overlap between hindcast data and measured data, and the only data in question are from 1987 (while previous hindcast data have been validated), the most effective approach to obtain a data set that best reflects all available information is to produce a composite data set with a combination of hindcast and measured data. The composite data set includes all of the Station 62 WIS hindcast data with the exception of those events after February 1987. The 1987 data will be replaced with data from the NDBC buoy 45007 or from the Burns Harbor pressure sensor if available. The NDBC buoy data are corrected according to the empirical relation between WIS Stations 64 and 62 so that it is equivalent to WIS Station 62 near Burns Harbor. The empirical relation between WIS Stations 64 and 62 is given by:

$$HS62 = HS64 \; \{1.162 - (0.026 * TP64)\} \tag{3-2}$$

where

$HS64$ = Station 64 significant height

$HS62$ = Station 62 significant height

$TP64$ = Station 64 peak period

Figure 3-6 displays the relation between HS64 and HS62.

Table 3-2 contains the correct (Station 62 equivalent) values and associated NDBC wave parameters for the 1987 data.

Along with these data an extreme value measured at Burns Harbor was added to the composite data. This extreme value was HS = 10.5 ft (3.2 m) and TP = 9.9 sec for 03/09/87. WIS Station 62 data with and without the above additions are listed in Appendix 3B.

For purposes of comparison, extremal analyses were computed for both WIS Station 62 and the composite data set. Graphical results of these two extremal analyses (Figures 3-7 and 3-8) reveal that there is little difference in the overall result. Figure 3-7 and 3-8 are similar because the very highest extremes were not affected by adding the corrected NDBC and Burns Harbor extremes and WIS Station 62 data from 1987 are so much smaller than measured extremes that they did not enter into the extremal analysis, which only includes Station 62 data. Essentially, the problem of possible errors in the 1987 hindcast data does not carry over into the extremal analysis. This was not, however, apparent before performing this analysis. The small differences in the following estimated return period significant heights are not large enough to be considered particularly significant.

The two fitted extremal models have the following form:

NDBC Station 62 data alone:

Model 1: $-ln(-ln(P)) = (1.796 * HS) - 6.171$

where

P = the cumulative probability distribution value associated with HS or with a given return period for which HS is desired

n = 58, the number of extreme waves (local maxima) in excess of 3 m for the 32-year hindcast

HS = significant height

NDBC Station 62 data with corrected NDBC and Burns Harbor extremes:

Model 2: $-ln(-ln(P)) = (1.847 * HS) - 6.3245$, ($n$ = 64)

The general formula for return period R is:

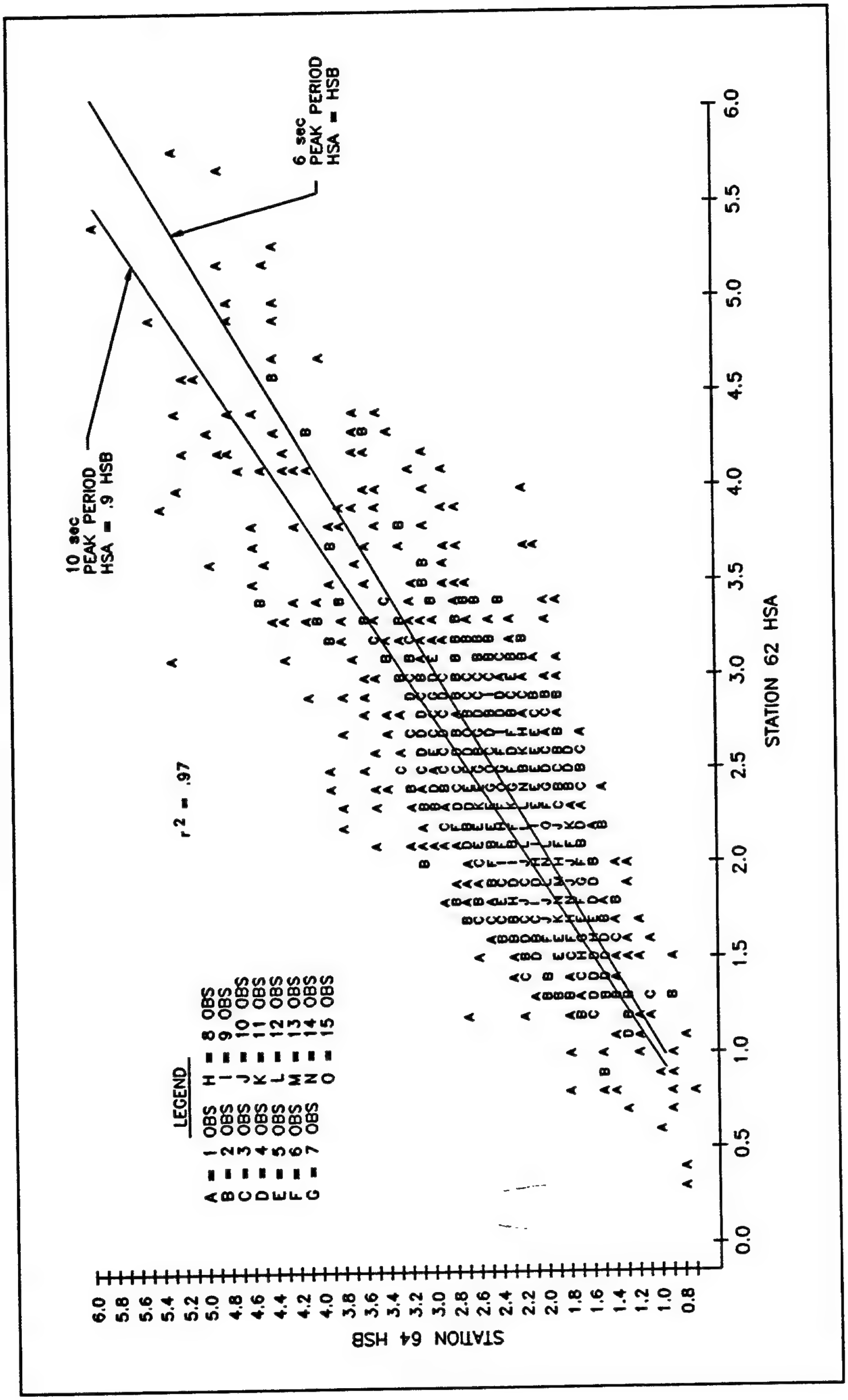

Figure 3-6. Plot of the relation between HS64 and HS62

$$R = \frac{1}{u(1 - P)} \tag{3-3}$$

where

u = local extremes per year (n/years either 58/32 or 64/32)

Table 3-2
Station 62 Equivalent Wave Heights

Date	NDBC Data Hs m	NDBC Data TP sec	HS62 Equivalent m
04/02/87	3.8	7.7	3.7
04/21/87	3.4	8.3	3.2
10/03/87	4.8	10.0	4.3
10/07/87	4.0	9.7	3.7
10/22/87	3.2	7.7	3.1

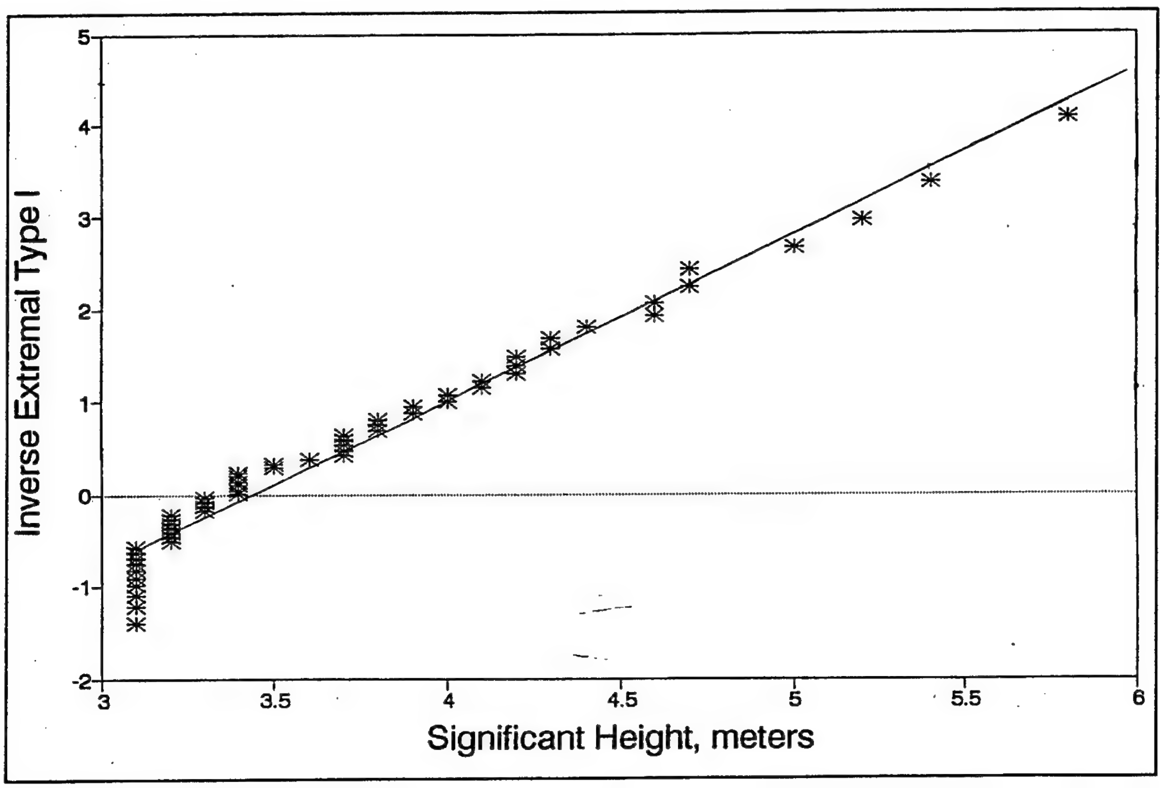

Figure 3-7. Extremal analysis of WIS Station 62 data

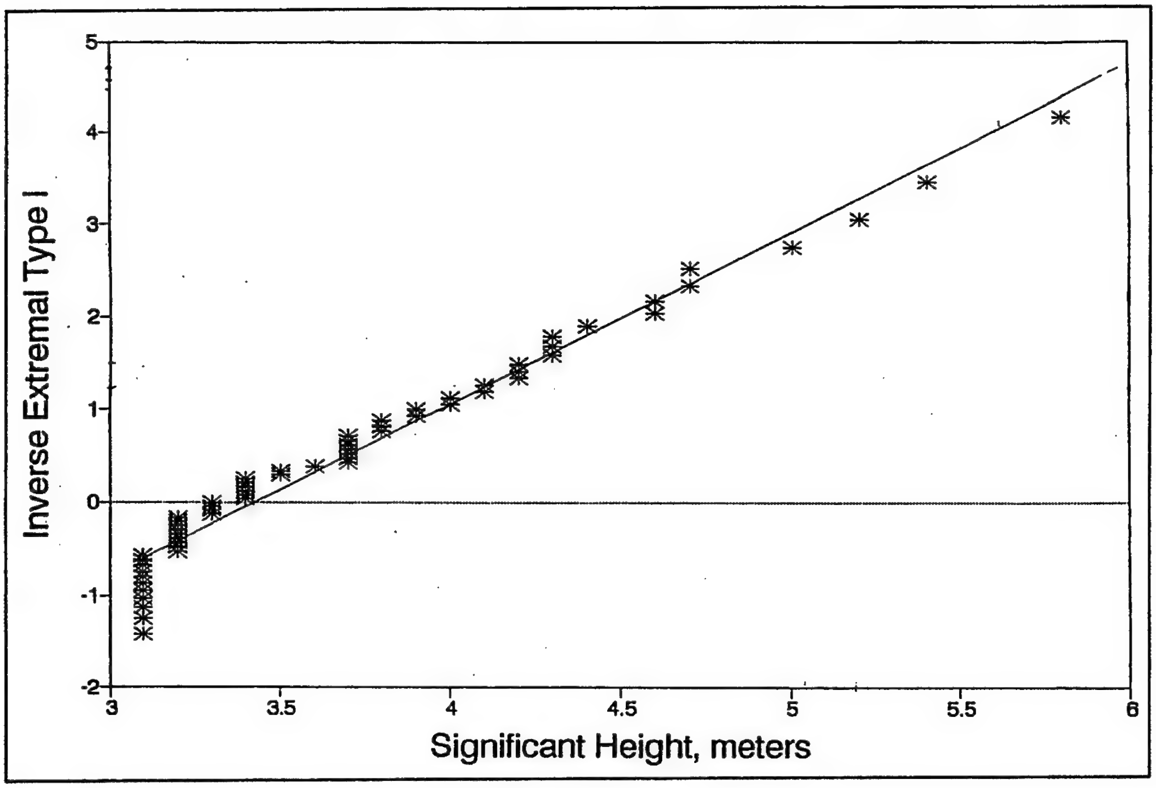

Figure 3-8. Extremal analysis of composite data set

Table 3-3
Design Wave Conditions

Return Period yr	Model 1 Significant Height m	Model 2 Significant Height m
2	4.07	4.10
5	4.63	4.64
10	5.03	5.03
20	5.43	5.39
32	5.69	5.67
50	5.94	5.91
100	6.32	6.29

Applying the formula for *P* and solving for *HS* using the appropriate model results in the design conditions of Table 3-3. Values of significant wave height versus return period are plotted in Figure 3-9 for both models. The estimates of Table 3-3 are reported to the nearest hundredth for purposes of demonstrating small differences between the two models. Accuracy to this level is not implied. It is interesting to note that the 50-year wave height reported by WIS for Station 62 is 19.4 ft (5.9 m). The fact that the extremal analysis performed by WIS and this extremal analysis produce nearly identical 50-year wave heights further supports the validity of the result.

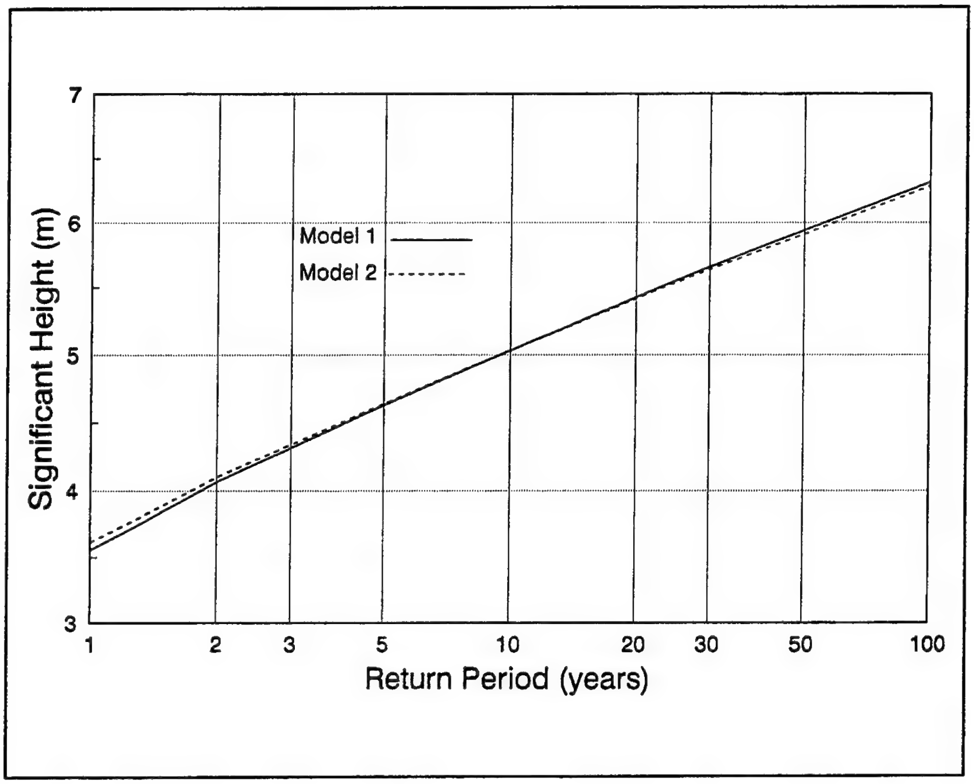

Figure 3-9. Comparison of significant wave height and return period

Discussion

This study was performed to provide verification of the WIS extremal analysis and to determine the effect, if any, of apparent disagreement between measured and hindcast data for the Burns Harbor area. The results of the study support the extremal analysis performed by WIS. The reliability of extremal predictions for any return period longer than 32 years is unknown and no claims as to the reliability or precision of these estimates are made by the author of this report.

Reference

Borgman, L. E., and Resio, D. T. (1982). "Extremal statistics in wave climatology," *Topics in Ocean Physics*, Society of Italiana di fisica, Bologna, Italy.

Appendix 3A
Raw Data from WIS Station 64 and NDBC Buoy 45007

		WIS STATION 64									NDBC BUOY 45007									
OBS	TIME	HSB	TPB	DIRB	WSB	WDB	DURB	STATION	DIRA	DC	NWS	NWD	NHS	NTP	DCN	PDIFF	DDIFF	WDIFF	SDIFF	CROSS
1	85040609	1.5	4.8	300	10	290	4	64	-	1	13	270	2.8	7.7	4	2.9	-20	1.3	3	8.7
2	85040612	1.9	5.6	303	10	280	4	64	-	1	14	270	2.8	5.9	4	0.3	-10	0.9	4	1.2
3	85040615	1.8	5.3	300	10	275	4	64	-	1	13	280	3.1	6.3	4	1.0	5	1.3	3	3.0
4	85040618	1.7	5.3	307	10	295	4	64	-	1	8	290	2.6	6.3	4	1.0	-5	0.9	-2	-2.0
5	85041012	1.7	5.3	195	11	200	6	64	-	1	11	200	2.0	5.9	3	0.6	0	0.3	0	0.0
6	85041018	2.3	6.2	195	11	195	6	64	-	1	12	190	2.2	6.3	3	0.1	-5	-0.1	1	0.1
7	85041618	1.8	5.3	10	12	5	4	64	-	1	13	0	2.2	6.7	1	1.4	-5	0.4	1	1.4
8	85041621	2.2	6.2	6	11	5	4	64	-	1	11	0	2.3	6.7	1	0.5	-5	0.1	0	0.0
9	85041700	2.3	6.7	8	10	10	4	64	-	1	11	10	2.2	7.1	1	0.4	0	-0.1	1	0.4
10	85051709	1.4	4.3	4	13	5	5	64	-	1	14	0	2.5	6.7	1	2.4	-5	1.1	1	2.4
11	85051712	2.0	5.6	3	13	5	5	64	-	1	13	0	3.1	7.7	1	2.1	-5	1.1	0	0.0
12	85051715	2.5	6.7	3	12	5	5	64	-	1	13	0	3.0	8.3	1	1.6	-5	0.5	1	1.6
13	85051718	2.2	6.7	4	10	5	5	64	-	1	10	0	2.3	7.7	1	1.0	-5	0.1	0	0.0
14	85073118	1.5	5.0	59	10	60	3	64	-	1	11	60	2.1	5.9	2	0.9	0	0.6	1	0.9
15	85092321	2.1	5.9	196	11	210	6	64	-	1	11	170	2.1	5.9	3	0.0	-40	0.0	0	0.0
16	85092400	2.6	6.7	228	14	250	6	64	-	1	16	250	2.6	6.3	4	-0.4	0	0.0	2	-0.8
17	85092403	2.2	5.9	257	11	270	6	64	-	1	13	270	2.5	6.7	4	0.8	0	0.3	2	1.6
18	85092406	1.6	5.3	262	9	255	6	64	-	1	13	270	2.4	5.9	4	0.6	15	0.8	4	2.4
19	85100506	1.9	5.9	250	10	270	6	64	-	1	14	270	2.3	5.3	4	-0.6	0	0.4	4	-2.4
20	85100509	2.0	5.9	279	12	295	6	64	-	1	14	280	2.5	5.9	4	0.0	-15	0.5	2	0.0
21	85100512	2.5	6.7	305	12	295	6	64	-	1	12	300	2.4	6.3	4	-0.4	5	-0.1	0	0.0
22	85100515	2.2	6.2	305	10	300	6	64	-	1	12	310	2.2	6.3	4	0.1	10	0.0	2	0.2
23	85100806	1.5	5.0	175	9	180	7	64	-	1	12	170	2.1	5.6	3	0.6	-10	0.6	3	1.8
24	85100809	2.0	5.9	182	11	180	7	64	-	1	11	180	2.2	6.7	3	0.8	0	0.2	0	0.0
25	85100812	2.4	6.7	188	11	190	7	64	-	1	7	200	2.5	7.1	3	0.4	10	0.1	-4	-1.6
26	85100818	2.1	6.2	197	10	200	7	64	-	1	11	190	2.4	6.7	3	0.5	-10	0.3	1	0.5
27	86060121	2.0	5.9	3	11	5	4	64	-	1	12	0	2.5	6.7	1	0.8	-5	0.5	1	0.8
28	86060200	2.5	6.7	5	11	5	4	64	-	1	12	0	2.4	7.1	1	0.4	-5	-0.1	1	0.4
29	86060203	1.8	6.7	9	9	10	4	64	-	1	9	10	2.0	7.1	1	0.4	0	0.2	0	0.0
30	86062415	2.1	5.9	6	11	5	3	64	-	1	11	0	2.1	6.3	1	0.4	-5	0.0	0	0.0
31	86082203	2.1	5.9	11	10	20	3	64	-	1	10	20	2.5	7.1	1	1.2	0	0.4	0	0.0
32	86091603	2.5	6.2	31	13	45	6	64	-	1	11	50	2.1	7.1	2	0.9	5	-0.4	-2	-1.8
33	86100415	2.0	5.9	53	10	70	6	64	-	1	9	80	2.1	7.1	2	1.2	10	0.1	-1	-1.2
34	86100421	2.3	6.2	358	12	355	6	64	-	1	10	350	2.2	6.3	1	0.1	-5	-0.1	-2	-0.2

		WIS STATION 64									NDBC BUOY 45007								
OBS	TIME	HSB	TPB	DIRB	WSB	WDB	DURB	STATION	DIRA	DC	NWS	NWD	NHS	NTP	DCN	PDIFF	DDIFF	WDIFF	SDIFF	CROSS
35	86100603	2.6	6.2	320	15	325	7	64	-	1	12	330	2.8	6.7	1	0.5	5	0.2	-3	-1.5
36	86100606	2.5	6.2	338	14	335	7	64	-	1	11	330	3.1	7.7	1	1.5	-5	0.6	-3	-4.5
37	86100609	2.6	6.7	339	11	335	7	64	-	1	9	330	2.6	8.3	1	1.6	-5	0.0	-2	-3.2
38	86100718	2.6	6.7	184	13	175	10	64	-	1	11	170	2.1	5.9	3	-0.8	-5	-0.5	-2	1.6
39	86101418	2.4	5.9	248	15	250	12	64	-	1	12	250	2.1	5.6	4	-0.3	0	-0.3	-3	0.9
40	86101500	2.6	6.2	253	16	260	12	64	-	1	13	260	2.5	5.9	4	-0.3	0	-0.1	-3	0.9
41	86101503	2.5	6.2	263	16	270	12	64	-	1	12	270	2.1	5.9	4	-0.3	0	-0.4	-4	1.2
42	86110900	2.8	6.7	202	15	205	18	64	-	1	12	200	2.2	6.3	3	-0.4	-5	-0.6	-3	1.2
43	86110903	3.0	6.7	208	16	220	18	64	-	1	13	220	2.2	5.9	3	-0.8	0	-0.8	-3	2.4
44	86110906	3.0	6.7	217	17	225	18	64	-	1	13	220	2.3	5.9	3	-0.8	-5	-0.7	-4	3.2
45	86110912	3.3	7.1	248	19	255	18	64	-	1	15	250	3.0	7.1	4	0.0	-5	-0.3	-4	0.0
46	86110915	3.0	6.7	251	18	260	18	64	-	1	14	260	3.1	6.7	4	0.0	0	0.1	-4	0.0
47	86110918	2.5	6.2	261	14	265	18	64	-	1	11	260	2.4	6.3	4	0.1	-5	-0.1	-3	-0.3
48	86110921	2.2	5.9	264	13	265	18	64	-	1	10	260	2.0	5.9	4	0.0	-5	-0.2	-3	0.0
49	86111000	2.0	5.6	266	12	265	18	64	-	1	10	260	2.0	5.6	4	0.0	-5	0.0	-2	0.0
50	86111300	2.6	6.2	300	14	300	11	64	-	1	11	300	2.3	6.7	4	0.5	0	-0.3	-3	-1.5
51	86111303	2.9	6.7	311	15	305	11	64	-	1	12	300	3.1	7.7	4	1.0	-5	0.2	-3	-3.0
52	86111306	2.6	6.7	313	13	310	11	64	-	1	10	310	2.5	7.1	4	0.4	0	-0.1	-3	-1.2
53	86111415	2.9	6.7	194	15	195	10	64	-	1	12	190	2.1	6.3	3	-0.4	-5	-0.8	-3	1.2
54	86111418	3.0	7.1	193	14	195	10	64	-	1	11	190	2.3	6.7	3	-0.4	-5	-0.7	-3	1.2
55	86111421	2.9	6.7	192	14	195	10	64	-	1	11	190	2.1	6.7	3	0.0	-5	-0.8	-3	0.0
56	86111821	2.6	6.2	29	14	20	6	64	-	1	11	10	2.1	6.3	1	0.1	-10	-0.5	-3	-0.3
57	86111900	2.9	7.1	8	14	5	6	64	-	1	11	0	2.4	6.7	1	-0.4	-5	-0.5	-3	1.2
58	86112000	2.6	6.2	146	13	135	8	64	-	1	11	130	2.2	6.3	2	0.1	-5	-0.4	-2	-0.2
59	86112003	2.5	6.2	145	13	140	8	64	-	1	11	140	2.3	6.3	3	0.1	0	-0.2	-2	-0.2
60	86112006	2.7	6.2	144	14	135	8	64	-	1	12	130	2.1	6.3	2	0.1	-5	-0.6	-2	-0.2
61	87040203	1.7	5.6	287	9	300	7	64	-	1	14	290	2.3	6.3	4	0.7	-10	0.6	5	3.5
62	87040206	1.9	5.9	309	9	315	7	64	-	1	13	300	2.8	6.7	4	0.8	-15	0.9	4	3.2
63	87040209	1.9	5.9	321	9	320	7	64	-	1	14	310	3.5	7.7	4	1.8	-10	1.6	5	9.0
64	87040212	1.8	5.6	322	9	315	7	64	-	1	13	310	3.8	7.7	4	2.1	-5	2.0	4	8.4
65	87040215	1.9	5.9	317	9	310	7	64	-	1	14	310	3.4	9.1	4	3.2	0	1.5	5	16.0
66	87040218	1.7	5.6	332	9	340	7	64	-	1	12	310	3.2	8.3	4	2.7	-30	1.5	3	8.1
67	87040509	1.6	5.3	5	9	5	5	64	-	1	12	10	2.2	6.7	1	1.4	5	0.6	3	4.2
68	87040512	2.1	5.9	4	10	5	5	64	-	1	12	0	2.6	7.1	1	1.2	-5	0.5	2	2.4

		WIS STATION 64									NDBC BUOY 45007									
OBS	TIME	HSB	TPB	DIRB	WSB	WDB	DURB	STATION	DIRA	DC	NWS	NWD	NHS	NTP	DCN	PDIFF	DDIFF	WDIFF	SDIFF	CROSS
69	87040515	2.3	6.7	6	10	10	5	64	-	1	12	30	2.8	7.7	1	1.0	20	0.5	2	2.0
70	87040518	2.6	7.1	10	11	15	5	64	-	1	9	20	2.4	7.7	1	0.6	5	-0.2	-2	-1.2
71	87040521	2.2	6.2	19	10	20	5	64	-	1	9	10	2.1	7.7	1	1.5	-10	-0.1	-1	-1.5
72	87042115	1.2	4.0	24	9	15	3	64	-	1	12	10	2.2	6.3	1	2.3	-5	1.0	3	6.9
73	87042118	1.5	5.0	14	9	15	3	64	-	1	12	10	2.9	7.7	1	2.7	-5	1.4	3	8.1
74	87042121	2.0	5.9	12	10	15	3	64	-	1	13	10	3.4	8.3	1	2.4	-5	1.4	3	7.2
75	87100118	1.2	4.3	206	9	195	13	64	-	1	12	190	2.1	5.9	3	1.6	-5	0.9	3	4.8
76	87100121	1.7	5.3	191	11	175	13	64	-	1	14	170	2.6	6.7	3	1.4	-5	0.9	3	4.2
77	87100200	1.8	6.2	186	9	180	13	64	-	1	11	180	2.1	6.2	3	0.0	0	0.3	2	0.0
78	87100203	1.8	5.9	193	9	200	13	64	-	1	11	200	2.3	6.7	3	0.8	0	0.5	2	1.6
79	87100209	2.1	5.9	295	12	290	13	64	-	1	15	290	2.3	5.9	4	0.0	0	0.2	3	0.0
80	87100212	2.4	6.2	301	12	300	13	64	-	1	14	300	2.8	6.7	4	0.5	0	0.4	2	1.0
81	87100215	2.6	6.7	321	11	320	13	64	-	1	14	320	3.1	8.3	1	1.6	0	0.5	3	4.8
82	87100218	2.7	7.1	330	11	330	13	64	-	1	14	330	3.4	8.3	1	1.2	0	0.7	3	3.6
83	87100221	2.8	6.7	339	12	340	13	64	-	1	15	340	4.4	8.3	1	1.6	0	1.6	3	4.8
84	87100300	2.9	7.1	349	12	350	13	64	-	1	15	350	4.8	10.0	1	2.9	0	1.9	3	8.7
85	87100303	3.2	7.7	359	12	0	13	64	-	1	15	360	4.7	10.0	1	2.3	360	1.5	3	6.9
86	87100306	2.4	7.7	352	10	345	13	64	-	1	12	340	3.5	10.0	1	2.3	-5	1.1	2	4.6
87	87100709	2.1	5.9	332	10	330	7	64	-	1	13	330	2.6	7.1	1	1.2	0	0.5	3	3.6
88	87100712	2.4	6.2	342	11	340	7	64	-	1	14	340	3.3	7.7	1	1.5	0	0.9	3	4.5
89	87100715	2.7	7.1	347	11	345	7	64	-	1	14	340	3.6	8.3	1	1.2	-5	0.9	3	3.6
90	87100718	2.3	7.1	348	10	345	7	64	-	1	12	340	4.0	9.1	1	2.0	-5	1.7	2	4.0
91	87102112	1.5	5.0	279	9	275	3	64	-	1	11	270	2.1	6.2	4	1.2	-5	0.6	2	2.4
92	87102218	1.5	5.0	168	10	170	3	64	-	1	12	170	3.2	7.1	3	2.1	0	1.7	2	4.2
93	87102221	1.5	5.0	170	9	180	3	64	-	1	12	180	3.2	7.7	3	2.7	0	1.7	3	8.1
94	87110509	1.5	5.0	311	9	310	4	64	-	1	12	310	2.2	5.9	4	0.9	0	0.7	3	2.7
95	87110512	1.7	5.6	317	9	310	4	64	-	1	11	310	2.3	6.7	4	1.1	0	0.6	2	2.2
96	87110515	1.9	5.9	312	9	305	4	64	-	1	12	300	2.7	6.7	4	0.8	-5	0.8	3	2.4

Appendix 3B
WIS Station 62
Extremes > 3 m

WIS STATION 62, Extremes > 3 m

YR\MO\DAY	HS M	TP SEC	COMMENTS
57121106	3.8	8.3	--
59011600	3.1	7.1	--
59012203	4.1	9.1	--
60011518	3.3	7.7	--
60021112	3.4	9.1	--
60032215	3.2	7.1	--
61012421	3.2	7.7	--
61022521	3.4	7.7	--
61030918	4.0	8.3	--
61112715	3.1	7.7	--
62013018	3.5	7.7	--
62022821	3.2	7.7	--
62041312	3.1	7.7	--
63032118	3.1	7.7	--
64011312	3.7	7.7	--
65022518	5.8	10.0	--
65122515	4.3	8.3	--
66112900	4.6	9.1	--
67012718	4.2	9.1	--
68031221	3.3	7.1	--
68121503	3.7	8.3	--
69032518	3.2	9.1	--
71112121	3.3	7.7	--
71113003	3.5	7.7	--
72010421	3.1	7.7	--
72021918	3.1	7.1	--
72032300	3.1	7.1	--
72111418	3.8	8.3	--
73012900	4.3	8.3	--
73021521	3.7	9.1	--
73031800	3.9	10.0	--
73121400	3.3	8.3	--
74022221	4.4	9.1	--
74120209	3.4	8.3	--
74120821	3.2	8.3	--
75101818	3.1	7.7	--
75111321	4.0	8.3	--
76010800	3.1	7.7	--
76020200	4.2	9.1	--
76022206	4.7	9.1	--
76122015	3.6	7.7	--
78020521	3.4	7.1	--
79010118	3.1	7.7	--
79011409	4.1	8.3	--
79022600	4.2	8.3	--
79122506	4.7	9.1	--
80120218	3.9	8.3	--

WIS STATION 62, Extremes > 3 m

YR\MO\DAY	HS M	TP SEC	COMMENTS
80122421	3.8	7.7	--
81112012	3.7	8.3	--
83011509	3.4	7.7	--
83111115	4.6	10.0	--
83111621	3.7	9.1	--
84022821	5.0	9.1	--
85010115	3.2	7.1	--
85021221	5.2	10.0	--
86012703	3.2	7.1	--
86022109	3.4	7.7	--
87020821	5.4	9.1	--
87030915	3.2	--	Burns Harbor extreme
87040212	3.7	--	NDBC data corrected to Station 62
87042121	3.2	--	NDBC data corrected to Station 62
87100318	4.3	--	NDBC data corrected to Station 62
87100718	3.7	--	NDBC data corrected to Station 62
87102221	3.1	--	NDBC data corrected to Station 62

4 Evaluation of Breakwater Settlement

by John Andersen[1]

Introduction

The primary purpose of this report is recalculate settlement of the Burns Harbor breakwater structure for comparison to the initial calculations made during the project design phase. Immediate and consolidation settlement were computed based upon assumptions consistent with known facts. Immediate settlement was expected to take place during the process of construction, and consolidation settlement was expected to take place partly during construction and partly in the period afterwards. The sum of the immediate and consolidation settlement should represent the upper limit of the observed settlement. Prior to construction, the natural foundation consisted of a surface layer of clay at some locations and sand at other locations. This material was underlain by sandy clay.

The breakwater design called for removal of a layer of soft clay up to 20 ft (6.1 m) thick from the upper part of the breakwater foundation, and replacement of the clay with sand to a specified depth which varied along the length of the breakwater.

Settlement which might have been expected for this design condition were calculated during this investigation. Settlement was also computed for the in situ case in which the soft clay was not removed. The last case considered was one in which 5 to 10 ft (1.5 to 3.0 m) of clay was washed back into the dredged foundation space prior to backfilling with sand.

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Geology of the Area

Environments of deposition at the margin of the Lake Michigan basin were extremely varied during the Pleistocene period. Lake levels fluctuated radically as water outlets were sporadically blocked by ice masses and opened by glacial meltback. Depositional evidence of the radical changes in water surface elevation is indicated by ancient abandoned shorelines several to tens of miles inland. In the vicinity of Burns Harbor, Indiana, ice masses occupying the lake basin encroached upon the shore zone forming at times small lakes (few to tens of miles across) between the ice masses and the land. These lakes received clay-laden sediment loads discharged from glacial melt rivers flowing within and on top of the ice. Glacier-discharged coarse sediments settled quickly to the lake bottoms; clays remained suspended for longer periods, but eventually settled forming annular varved, lacustrine clay layers. The layers of clay were interrupted laterally by grounded ice masses calving from the ice fronts (icebergs) and stranded ice block remnants resulting from glacial meltback. Glacial meltback and shoreline sediment transport processes filled the depressions left by melting blocks of ice with clay layers interrupted laterally by sand zones. Clay layers were also interrupted during deposition by river sediment deposits extending lakeward (deltas) from land-based river systems.

The resulting deposits of lacustrine clays in the Burns Harbor area consist of clay interrupted by sand plugs or lenses. The sand plugs or lenses may vary in diameter due to the size of the ice block meltouts and/or river deltas. These sand plugs are highly permeable conduits with respect to the clays and are often the location of freshwater springs. The sources of the spring waters are deeper aquifers, located approximately 40 to 50 ft (12.2 to 15.2 m) beneath the clay/lake water interface.

Holocene and recent deposits associated with processes of land erosion and shoreline transport have capped the Pleistocene lacustrine clays with primarily sand material. Although site-specific details are not available for the immediate Burns Harbor area, similar geologic conditions prevail up and down the Lake Michigan shoreline and are expected in the Burns Harbor area.

Subsurface Conditions

Indiana Port Commission (1966b) describes the presence of three soil layers. The length of the breakwater is about 5,800 ft (1,769 m) with varying foundation conditions. The top stratum varied between 0 and 8 ft (0 and 2.4 m) and consisted of fine and medium sand with silty sand pockets. Reportedly this layer was not tested in the laboratory and only some Standard Penetration Test (SPT) data were available. This layer was underlain by gray, soft silty clay with some gravel at the bottom of the layer. The thickness of this layer did not exceed 20 ft (6.1 m) at any location. The bottom layer was glacial till consisting of gray, stiff silty clay occasionally mixed with sand and

gravel. This material prevailed to the maximum exploration depth. The three soil layers were not represented in all cross sections of the breakwater.

Settlement Calculations in the Breakwater Design

The original breakwater settlement evaluation prior to construction is given in documents prepared by the Indiana Port Commission (1966a). In that study, the upper limit of the settlement, related to consolidation for the top 13 ft (4 m) of the breakwater foundation, was found to be approximately 30 in. (76 cm). Foundation layers below 13 ft (4 m) were not evaluated for settlement, because the layers were predominantly sand or till and the settlement occurring in these layers would be immediate and/or small.

An assumption was also made that the rate of consolidation would be rapid and most of the consolidation would occur in a 6-month period. Replacement of the soft clay with sand was not considered in the original design computations prepared for the Indiana Port Commission (1965). Indiana Port Commission (1965) addresses the whole structure consisting of the harbor mooring facility and the breakwater. The consultant, Dr. Leonards, points out that no high quality undisturbed samples were obtained during the foundation exploration. Underlaying clays were grouped into two categories: a soft clay stratum at the top and a stiff clay stratum at the bottom. By evaluating preconsolidation pressures resulting from consolidation tests, Dr. Leonards concluded that the soft clay stratum was layered and probably contained lenses or pockets of alternating stiffer and softer material and suggested that this fact must be considered in settlement analyses. It was also his suggestion that the top soft clay, in case it was not removed, must be protected by a 5-ft- (1.5-m-) thick blanket of sand to prevent penetration of the coarser material into the softer foundation layer during the process of construction.

Dr. Leonards considered the possibility of uneven settlement and did not believe that such a case would be critical. The time rate of consolidation was impractical to estimate because of 3-D consolidation, unknown boundary conditions and very rapid consolidation in the soft clay. No breakwater section was anticipated to be completed in less than 6 months, thus allowing the consolidation. In Dr. Leonards' opinion, if shear distortion was involved in the deformation process of the breakwater foundation, the maximum amount of settlement would not exceed 2.5 ft (0.8 m). Dr. Leonards estimated the maximum longitudinal distortion of the breakwater to be less than 1/225 of longitudinal distance and not significant to breakwater performance. Indiana Port Commission (1966b) stated that maximum settlement would be on an order of 1 to 2 ft (0.3 to 0.6 m).

Geotechnical Information

Of the borings shown in Indiana Port Commission (1966d), 14 were taken at the breakwater location. Other borings described in the same reference were located elsewhere. Table 4-1 shows the data obtained from the

14 borings performed at the site of the breakwater. During the initial investigations, consolidation tests were performed on 12 samples obtained from the breakwater foundation. A Standard Penetration Test (SPT) was also performed in all of the breakwater borings (B22 through B34-2) to a depth 50 ft (15.2 m) below the lake bottom. SPT blow counts were the only data available at all depths in all borings. Atterberg limits were obtained on all samples and results are presented in Indiana Port Commission (1966d, 1966e).

The available references lacked any information on time-consolidation parameters. United States Naval Facilities Engineering Command (1986) indicates that the coefficient of secondary consolidation C_α would range from 0.008 to 0.012 since in situ water content for the cohesive soils in the breakwater foundation ranged between 28 and 58 percent.

Figure 4-1 shows the longitudinal profile of the breakwater foundation and displays some soil parameters. The same figure provides other pertinent information: schematic division of soils in three layers, as assumed by the designer of the breakwater, depth of excavated and replaced soil, and location of borings. Data used as sources of consolidation parameters are listed in Table 4-1. A general offshore profile of the breakwater and foundation conditions is shown in Figure 4-2 and boring locations are shown in Figure 4-3.

Figure 4-4 shows the relationship of the dry unit weight of soil with elevation. The median dry unit weight of the soils is about 100 pcf (1,600 kg/m^3). Figure 4-5 shows the relationship of the in situ water content of soils with elevation. In situ water content of the soils ranges between 20 and 44 percent, with maximums recorded in elevations between 528 and 548 ft (161 and 167 m). Figure 4-6 is a plot of in situ void ratio versus elevation. The void ratios ranged between 0.6 and 1.4, with maximum void ratios recorded on samples from elevations between 528 and 540 ft (161 and 165 m). Figure 4-7 gives the relationship between elevation and preconsolidation pressure as obtained from consolidation tests. Figure 4-8 shows the relationship of Atterberg limits to depth. The plasticity index (PI, the difference between liquid and plastic limit) ranges between 6 and 20 except for one instance where the PI reaches 60 in samples from elevations between 532 and 539 ft (162 and 164 m). Figure 4-9 shows the relationship of dry unit weight to soils and water content. Figure 4-10 presents the unconfined compression strength of soil samples versus elevation. The maximum value was recorded at the elevation of 530 ft (162 m). Additionally, Figure 4-11 depicts variation of compression index as a function of elevation and Figure 4-12 the relationship of water content and unconfined compression strength. The soils with higher content of in situ water have a marked decrease of strength.

Table 4-1
In Situ Properties of Soil

Boring No.	Elevation ft	Soil Classification[1]	Compression Index Cc	Recompression Index Cr	Liquid Limit (LL) %	Plastic Limit (PL) %	In Situ Void Ratio Eo	Water Content Wo, %	Preconsolidation Pressure Pc, kg/cm^2	Dry Unit Wt. γd, pcf	Unconfined Compression Uc, psi	Triaxial Test psi	Triaxial Test φ°	Soil Description
B-21	542.1	CL	.33	.04	30	15	.895	34.2	2.6	98.3	5.7			Gray soft moist silty clay
B-21	542.1	CL	.33	.04	31	16	.895	34.2	2.6	102.7	9.1			Light gray medium stiff clay of medium plasticity
B-22	536.5	CH	.29	0	52	25	1.225	42.4	4	81.4	7.7			Gray wet medium stiff clay
B-23	528	CH	.331	.05	74	27	1.207	44.6	.5	84	2.25			Gray wet very soft clay
B-24	520.2	CL-ML	.182	0	33	16	.742	27.6	.43	94.6		3	25	Gray clayey silt
B-26-2	535.6	CH	.366	.06	59	24	1.192	44.7	1	79.4	9.7			Gray moist stiff silty clay
B-26-2	529.6	CL	.425	0	28	15	1.357	27.2	.4	110	18.5			Gray stiff very silty clay with some gravel
B-28	520.65	CL	.158	.018	28	15	.647	23.9	.71	102.2	2.1			Gray wet very soft clay with trace of coarse sand
B-30	508.15	CL	.183	.014	26	15	.61	22.3	2.05	109.2				Not available
B-32	528.5	CL	.053	0	32	15	.69	25.2	.28	103.6	3.56			Moist medium soft gray silty clay
B-34-2	531.4	CL	.203	.019	25	18	.606	22.3	1.8	116.1	23.5			Gray moist soft silty clay
B-34-2	565.4	CL	.173	.02	36	16	.738	26.3	.95	107.4	19.6			Gray moist stiff silty clay

CH = Inorganic clay of high plasticity.
CL = Inorganic clay of low medium plasticity, sandy clay, silty clay, lean clay.
MH = Inorganic silt, very fine sand, silty clayey sand, clayey silt with plasticity.
Pc = Preconsolidation pressure.
Wo = In situ water content percent.
Eo = In situ void ratio.
PL = Plastic limit.
LL = Liquid limit.
Cr = Recompression index.
Cc = Compression index.
1 = United Soil Classification System, Technical Memorandum No. 3-357, USAE WES, 1960.

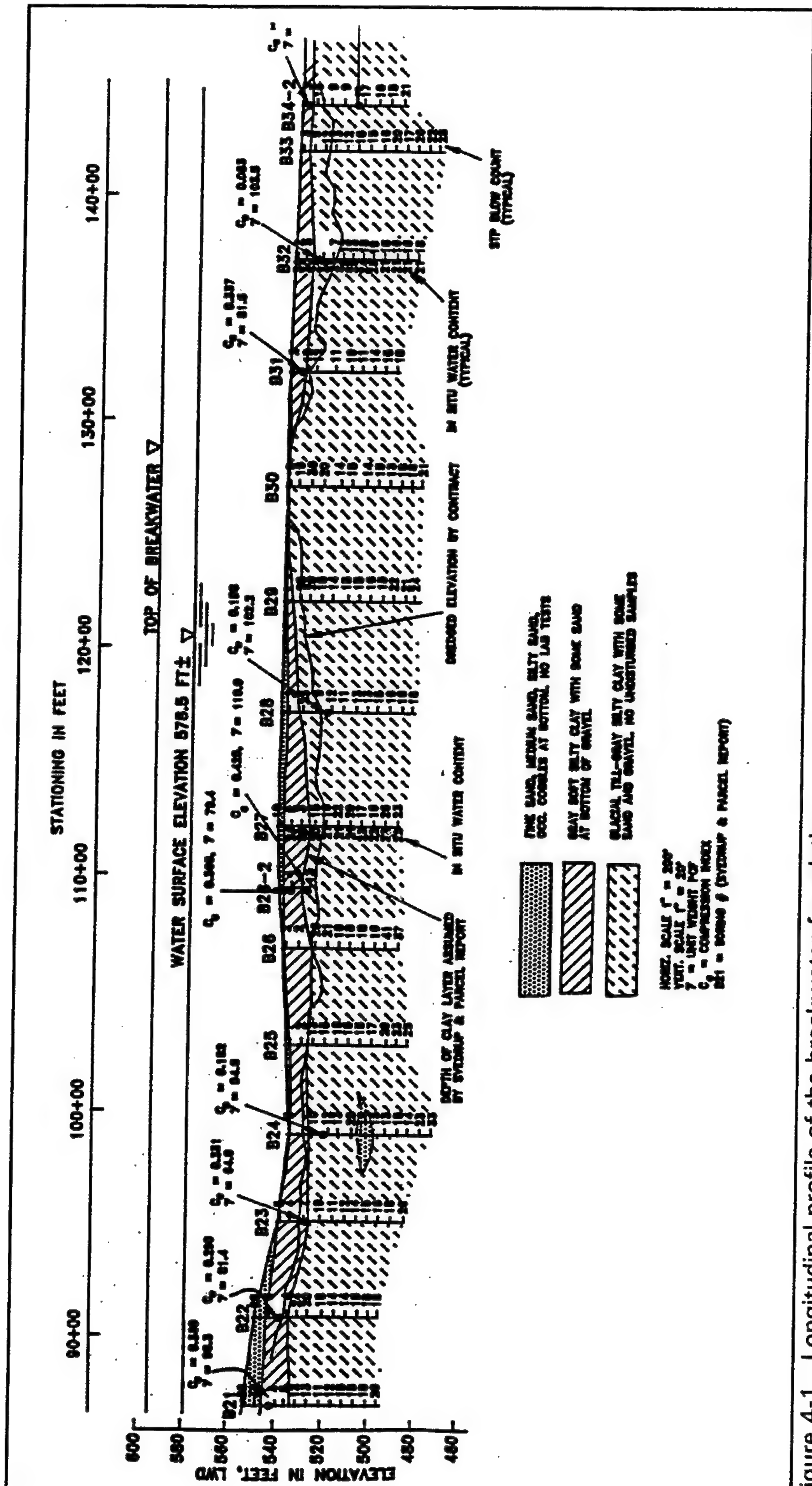


Figure 4-1. Longitudinal profile of the breakwater foundation

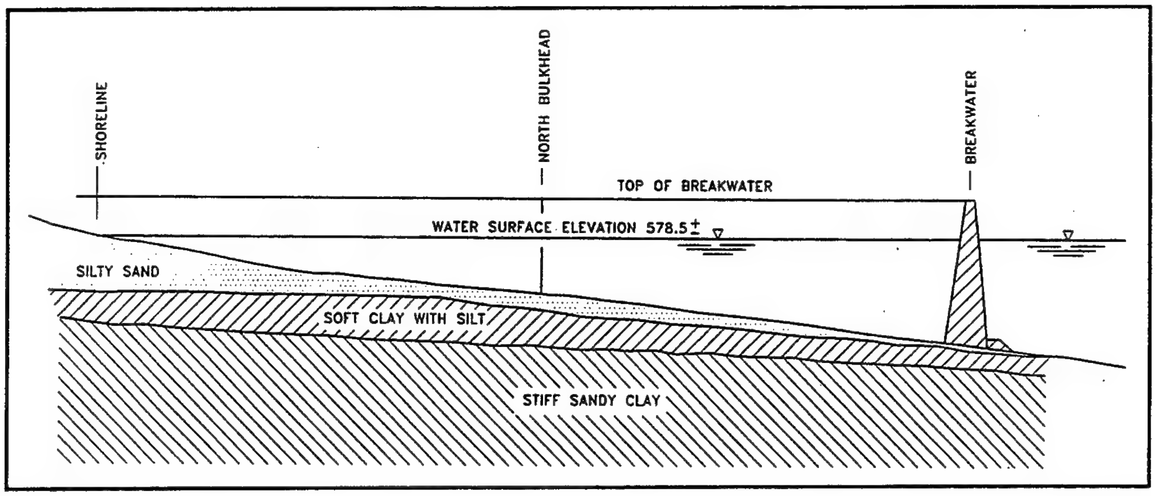


Figure 4-2. Generalized profile offshore (Indiana Port Commission 1966a)

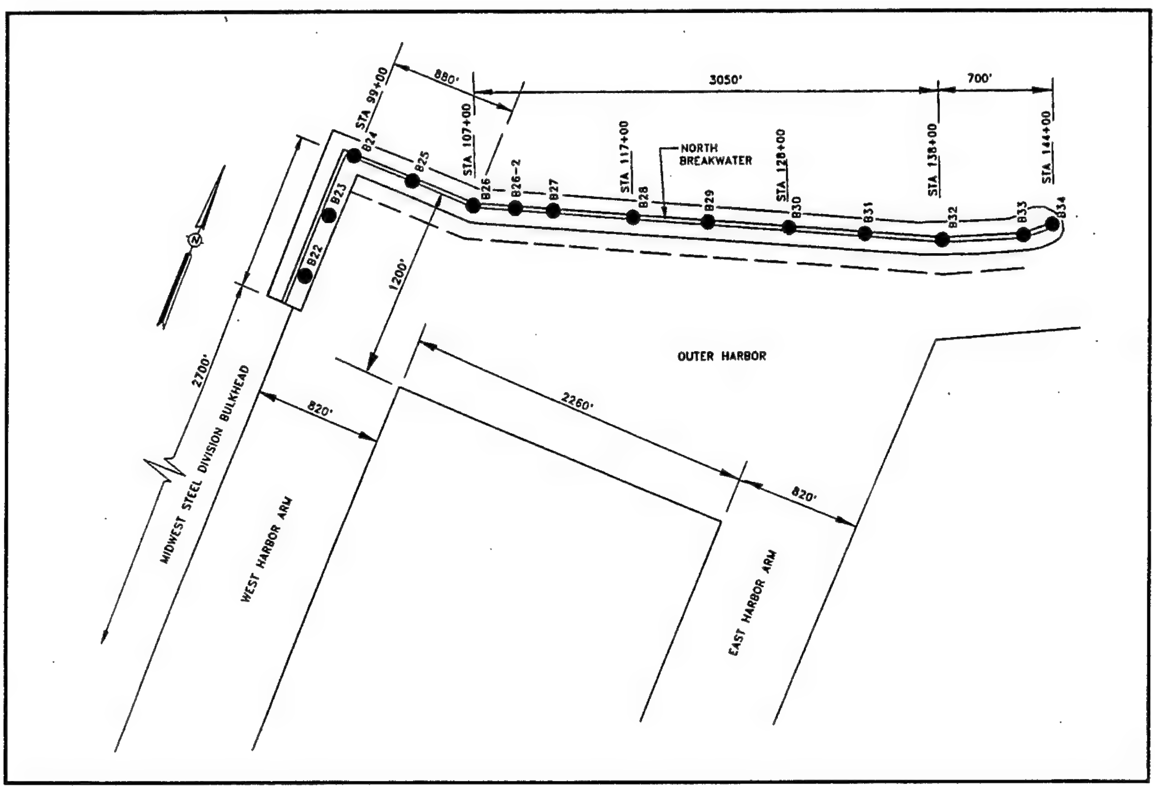


Figure 4-3. Location of borings

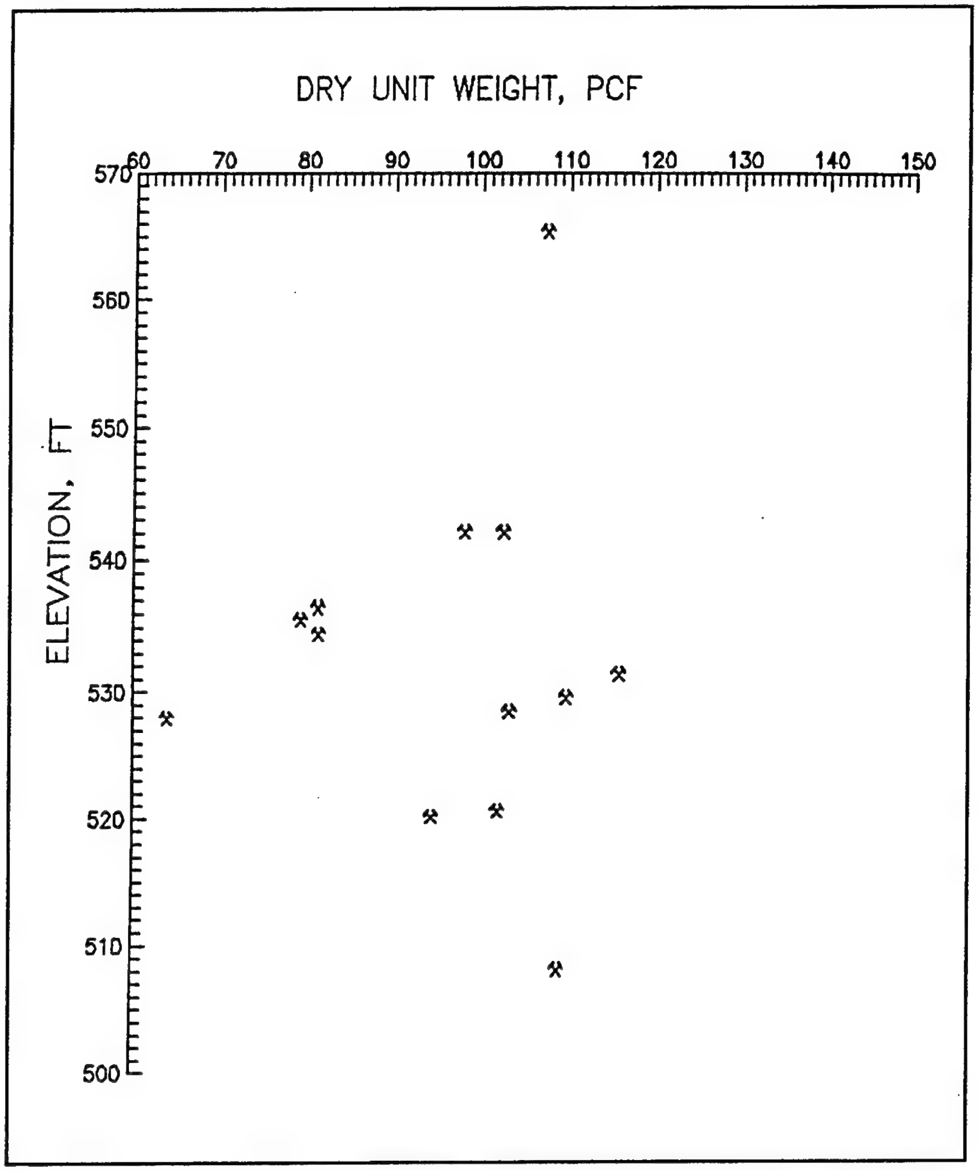


Figure 4-4. Dry unit weight of foundation soils (Indiana Port Commission 1966a)

Settlement Calculations[1]

Procedures

Vertical stresses in the foundation soils were computed using 5-ft- (1.5-m-) thick increments to a depth 30 ft (9.1 m) below the lake bottom and then in increments of approximately 30 ft (9.1 m) to a depth of 150 ft (46 m). Vertical stresses were then computed in increments of 150 ft (46 m). The increase of stress due to surcharge from the breakwater was considered to a depth of four widths (of the base of breakwater) which was about 600 ft (183 m). The increments of stress were computed using the method outlined in Lysmer and Duncan (1969).

A typical cross section of the breakwater is shown in Figure 4-13. The unit weight of riprap material constituting the upper portion of the breakwater was considered to be 140 pcf (2,243 kg/m^3). The void ratio of this riprap was considered 0.5, which is perhaps very conservative. The sandy portion of the breakwater above the foundation was considered to have a saturated density of 130 pcf (2,082 kg/m^3). The base width of the breakwater at the foundation varied from 152 to 188 ft (46 to 57 m) because the depth of the lake bottom varied. To compute surcharge stresses the cross section was considered constant at four typical widths: 157, 160, 162, and 171 ft (48, 48.8, 49.4, and 52.1 m). The surcharge exerted by the breakwater on the foundation was 3,500 psf (167,581 Pa) (of effective stress) at the center of the contact area. Because the surcharge was trapezoidal, the horizontal and vertical stresses in the subgrade also varied in distance from the center. Under the edge of the breakwater maximum vertical stresses did not occur at the interface of the breakwater and foundation, but at a depth of about one half of the width of structure.

Large structures exert stresses to substantial depths. Regardless of the materials in the deeper strata, this has to be taken into consideration. The properties of soils at depths below 50 ft (15.2 m) under the bottom of the lake were considered the same as those at the depth of 50 ft (15.2 m). This conservative assumption leads to an overestimation of settlement.

In situ and as-designed

Settlements were computed at sections coinciding with the boring locations: B24 (station 99+00), B26 (station 107+00), B28 (station 118+00), B30 (station 128+00), B32 (station 138+00), and B34 (station 144+00).

Moduli of elasticity for computation of immediate settlement by the Schmertman method (Schmertman, Hartman, and Brown 1978) were derived from the SPT results using procedures suggested by Bowles (1988).

[1] Water elevation in the Indiana Port Commission (1965) report figures was referenced to low water datum (LWD), which is +578.5 ft mean tide New York (MTNY 1935). For computations in this report, the International Great Lakes Datum (IGLD) which is 576.8 ft MTNY was taken from a U.S. Army Engineer District, Chicago 1975 survey.

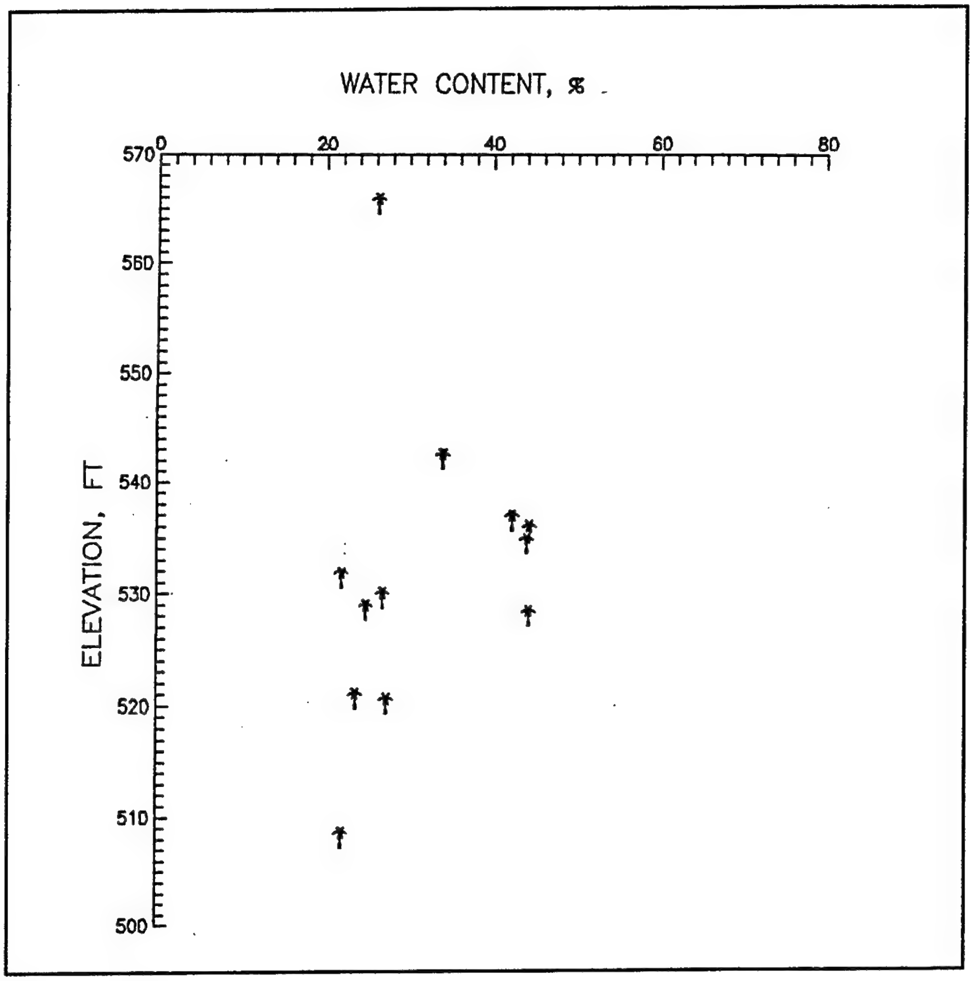


Figure 4-5. Water content of in situ foundation soils (Indiana Port Commission 1966a)

SPT blow counts were not reduced for depth because the soils were submerged. Parameters for the consolidation analyses were taken from consolidation test results (Indiana Port Commission 1966d). Table 4-2 and Figure 4-14 show the breakwater settlement computed for the in situ and as-built cases.

Reentry hypothesis

Because the backfilling of some sections was interrupted by winter, some of the excavated material may have reentered the open trench due to wave and current action. This hypothesis will be examined to estimate its potential impact on settlement.

Table 4-2
Calculated Settlement

Breakwater Station	In situ Condition (ft)	As-built Condition (ft)
99+00	2.51	1.96
107+00	2.50	1.41
117+00	2.63	1.47
128+00	1.50	1.50
138+00	2.61	1.25
1144+00	2.44	1.60

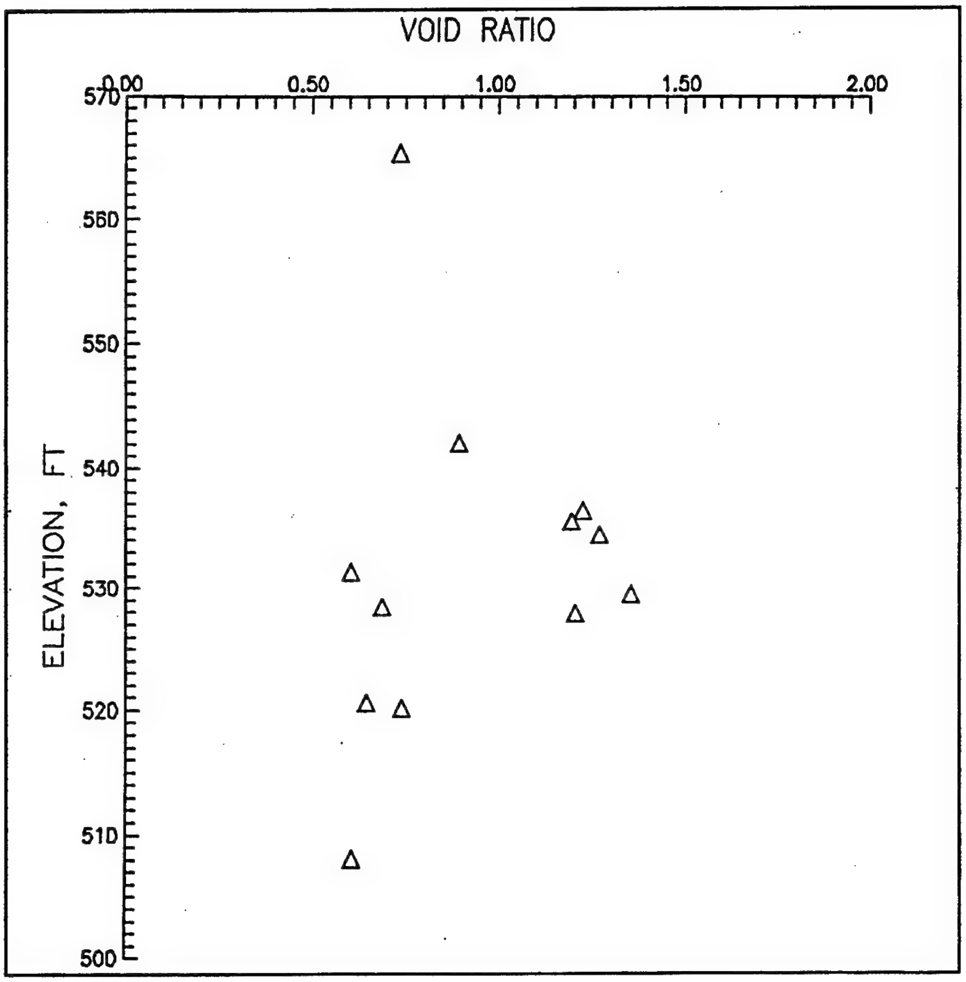


Figure 4-6. Void ratio of foundation soils (Indiana Port Commission 1966a)

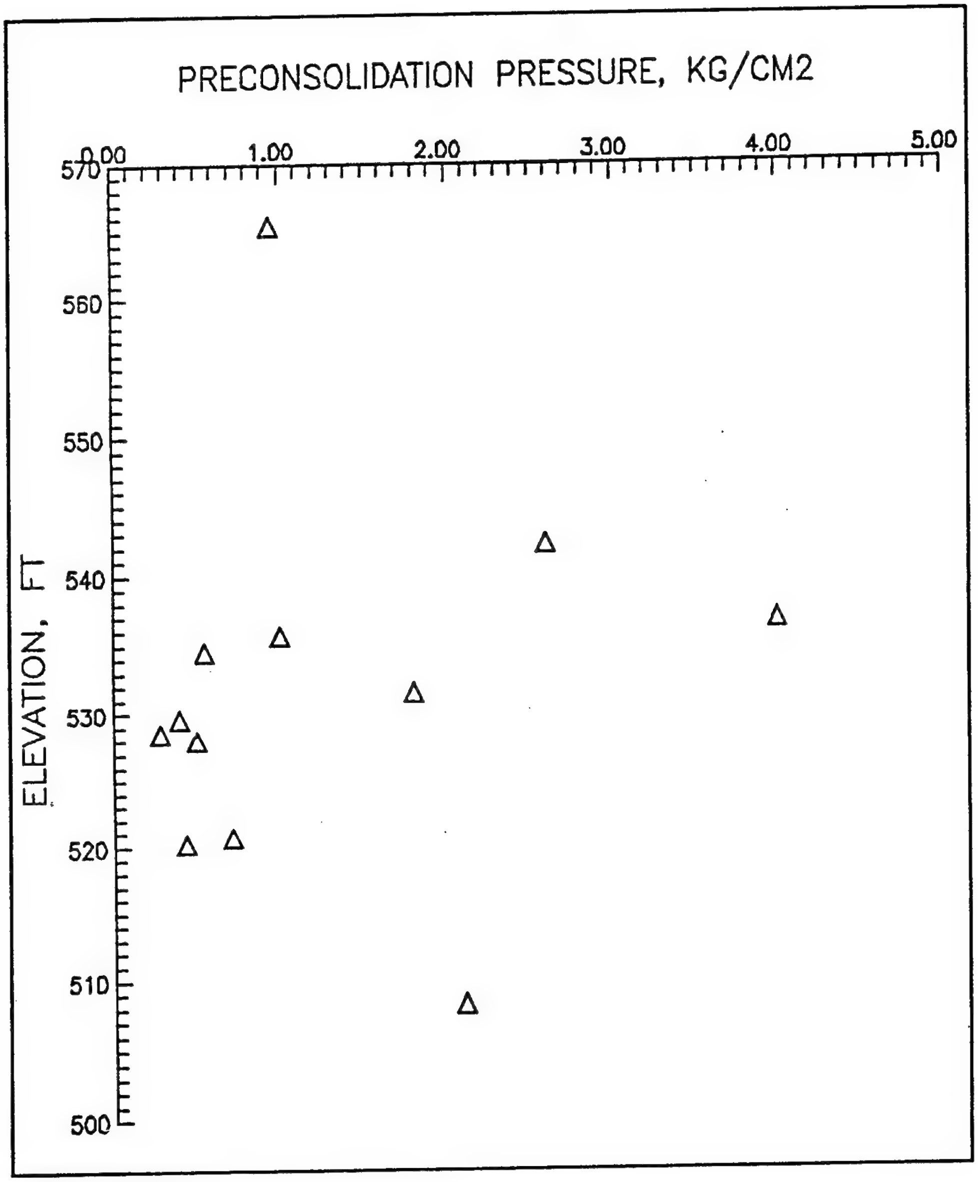


Figure 4-7. Preconsolidation pressure of foundation soils (Indiana Port Commission 1966a)

The effect of excavating the clay, dumping it outside the excavated area and having some of the clay redeposited back into the trench by current or wave action is assumed to constitute "remolding." Studies of remolded clays listed in American Society for Testing and Materials (1985) indicate that the compression indices of remolded clays are slightly lower than those on undisturbed clays. Actually the consistency of such material is difficult to represent in single form. Probably, it would be a mixture of lumped chunks and scattered smaller particles of clay. Skempton (1944) defined a correlation between liquid limit and compression index of remolded clays, and stated the relation between the undisturbed and remolded compression index is 1.3:1. Estimation of the consolidation parameters for clay, which underwent the

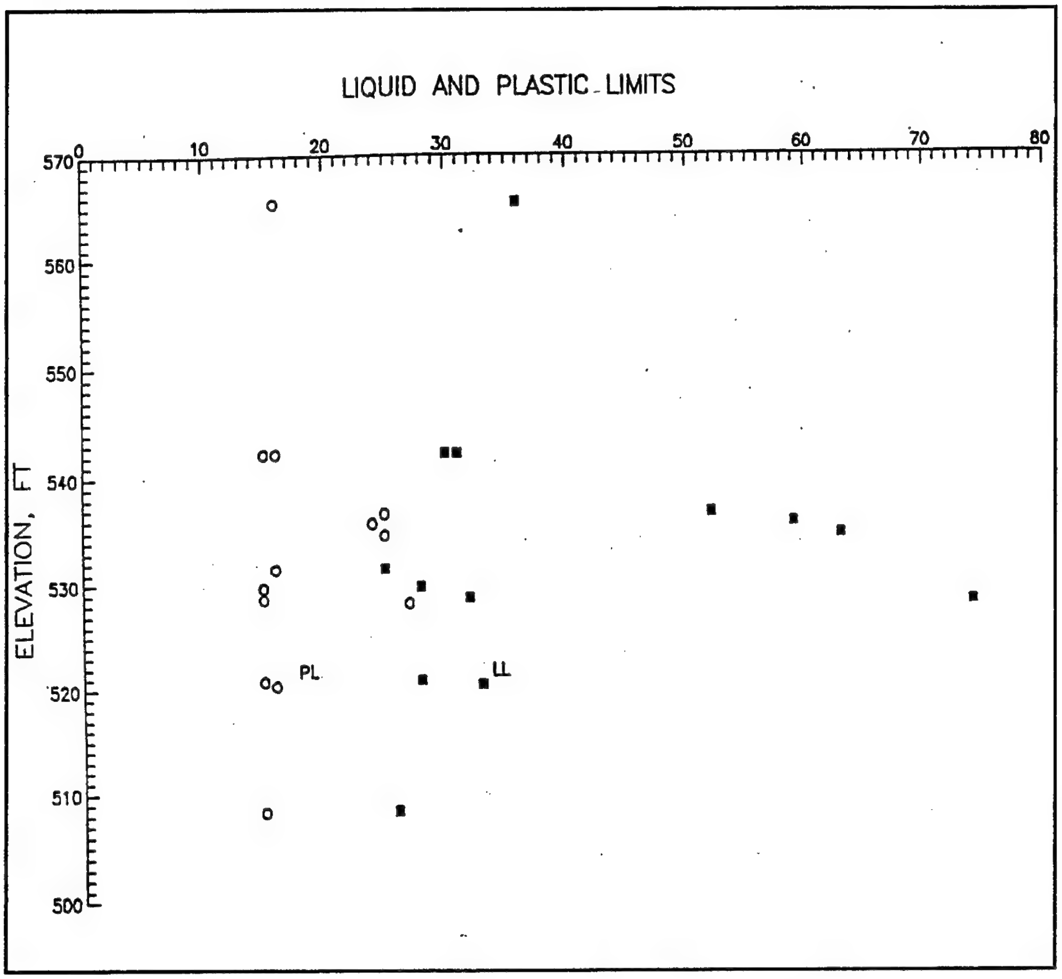


Figure 4-8. Atterberg limits of foundation soils versus depth (Indiana Port Commission 1966a)

process of dredging, dumping, and an uncertain mode of redeposition, is difficult. An estimation procedure was selected from Peters et al. (1996), in which two charts were developed for 23 kinds of soils that allowed for finding the remolded compression index and the position on e/log p curve for the new remolded initial void ratio. A procedure for calculating the settlement of disturbed clay is presented in Appendix 4A.

It is believed by the author that this procedure may come closer to the situation which occurred in construction of the breakwater. In Figure 4-15 the soils were plotted on a Plasticity chart with an identification number for each different soil type and source. Figure 4-16 (also developed by Peters et al. (1996)) gives the void ratio at 1 tsf on the ordinate and compression index on the abscissa. The numbered points in the graph correspond with the numbered points in Figure 4-15. Since the clays in the breakwater foundation exploration were tested for Atterberg limits, it is possible to plot the evaluated clay on Figure 4-15 and determine which of the known soils the soil in question resembles most. By selecting the nearest soil type from Figure 4-15, the void ratio and compression index for the remolded clay may be estimated.

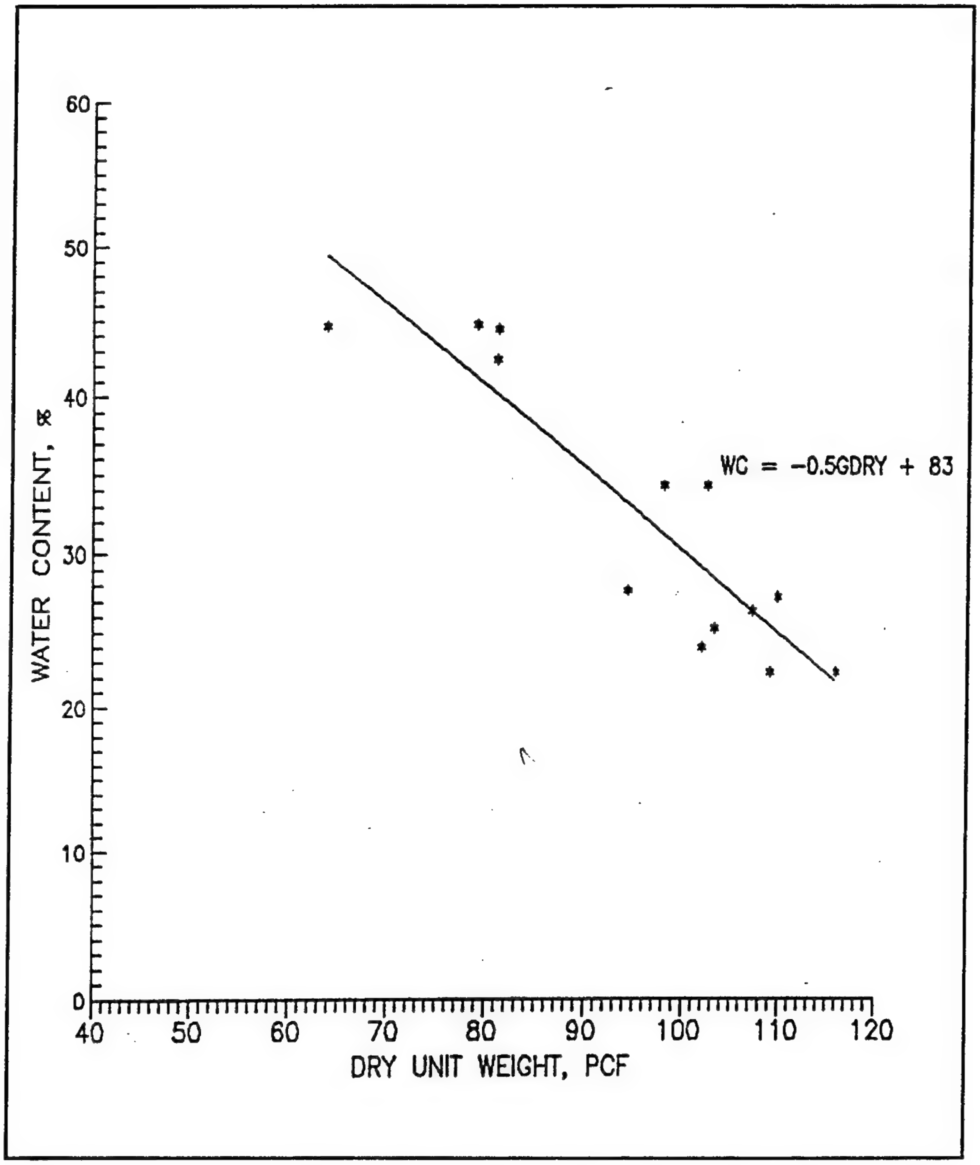


Figure 4-9. Relationship of water content to dry unit weights of soils

The sequence of the procedure is elucidated in Figure 4-17. Table 4-3 sums up data obtained from the tests and data derived from this procedure in Figure 4-17. The Atterberg Limits were selected from ranges of the existing available data.

This procedure is based on several assumptions since the history of the soil movement from its original excavated location to the final redeposition is unknown. It is not known whether the redeposited clays were mixed from several layers. It was assumed that soils excavated from the nearest borings were probably redeposited to the same location. In assuming several soils from various depths at the same location, it is impossible to say with any level of confidence whether the lower clay ended up at the top or lower elevation. Consequently, relatively conservative parameters were chosen for the disturbed soils. For comparison, data from Skempton (1944) and Peters et al. (1996) are presented in Figure 4-18 and Table 4-3.

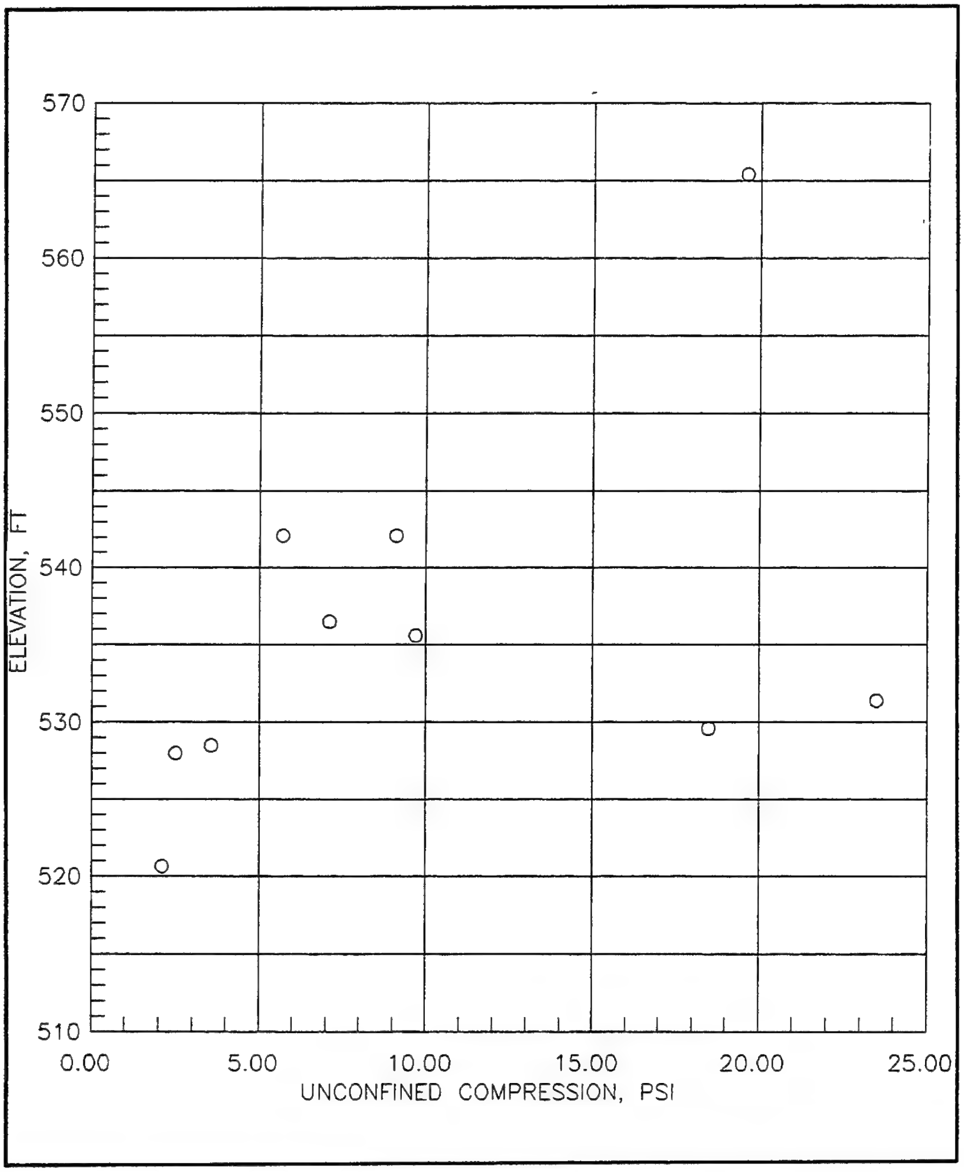


Figure 4-10. Unconfined compressive strength of foundation soils (Indiana Port Commission 1966a)

Discussion of Results

Three cases of settlement are depicted in Figure 4-19: one case is that all clay was removed and no clay was backfilled; the other two cases plot the contours of probable settlement if 5 or 10 ft (1.5 or 3.0 m) of clay was incidentally returned to the pre-excavated trench and topped with backfilled sand. Settlements range between 1.5 and 2.5 ft (0.5 and 0.8 m). Figure 4-14 shows the settlements for the design condition (all clay removed and foundation replaced by sand). The range of settlement is between 1.0 and 2.0 ft (0.3 and 0.6 m). Figure 4-14 also shows the "in situ condition" if the soils were left unexcavated and no replacement of any foundation soil took place.

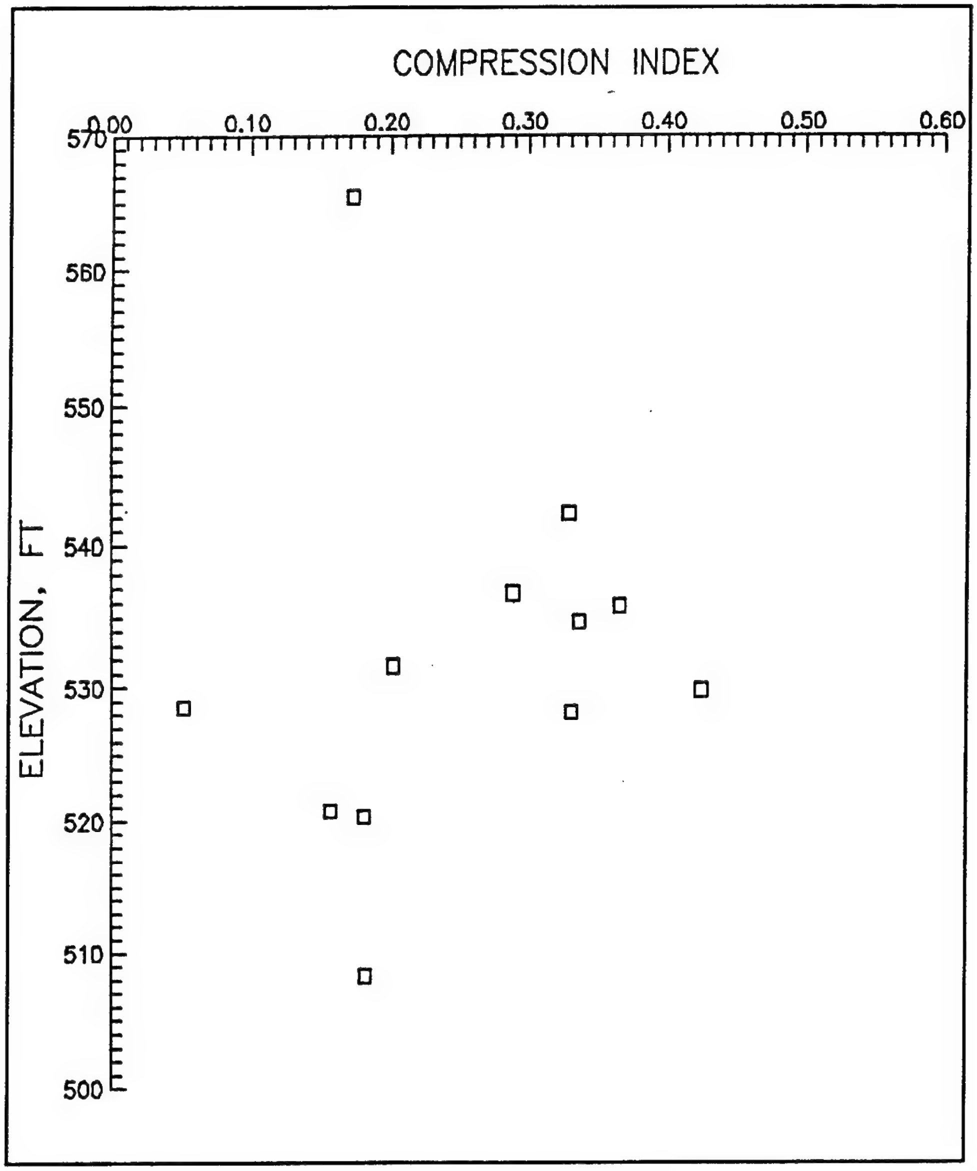


Figure 4-11. Compression indices versus elevation (Indiana Port Commission 1966a)

The settlements in this report are comparable with the settlements estimated during original design of the breakwater in Indiana Port Commission (1966c). Settlements obtained from Indiana Port Commission (1966c) are slightly higher because the consolidation was assumed to occur over a greater depth.

Figure 4-20 compares the changes in the breakwater top elevation after the first 8 years of its existence with the depth of excavated clay and the computed settlements for the as-built condition. Changes in the top elevation of the breakwater are represented by the survey data taken along the breakwater center line and by a polynomial curve fitted to the survey data.

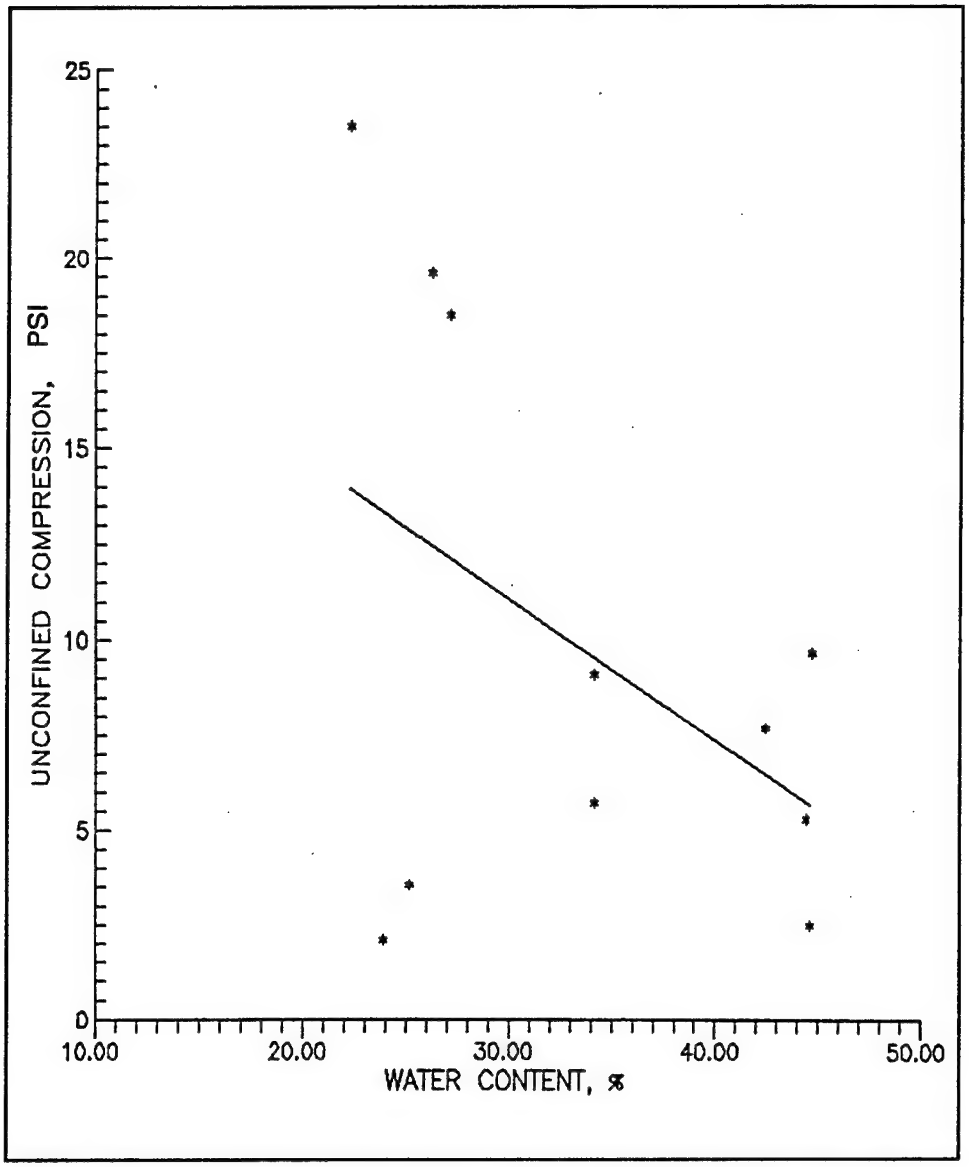


Figure 4-12. Water content and unconfined compression strength relationship (Indiana Port Commission 1966a)

The polynomial interpolation line indicates that minimum deflection on top of the breakwater along its center line coincided with the locations of maximum clay removal. This conjuncture is somewhat sensitive to the selection of the degree of the polynomial and therefore is regarded as inconclusive. Since the breakwater top elevations varied radically between adjacent cross sections, a much more involved method would be required to definitely establish whether a significant trend or relationship between these two parameters actually exists. The onset of the settlement on any structure coincides with the beginning of construction when the foundation soils are first surcharged. Typically for a construction process lasting several months on a sandy foundation, the major portion of the settlement is completed during the construction process. The total settlement consists of three portions: the most rapid settlement is the elastic settlement, which occurs immediately. Such settlement would likely result in additional material being furnished by the contractor. The second portion is primary consolidation, which represents

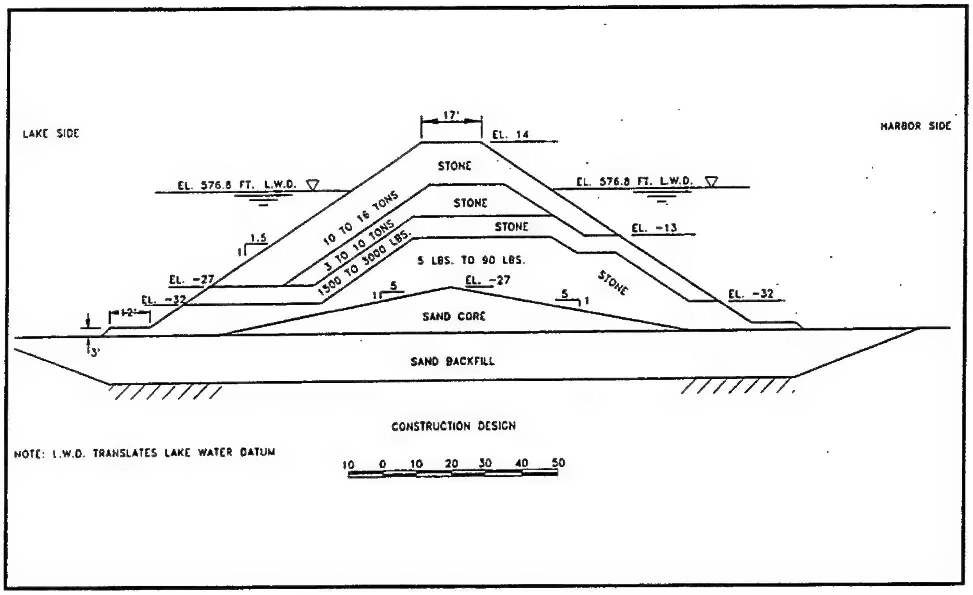


Figure 4-13. Typical cross section materials

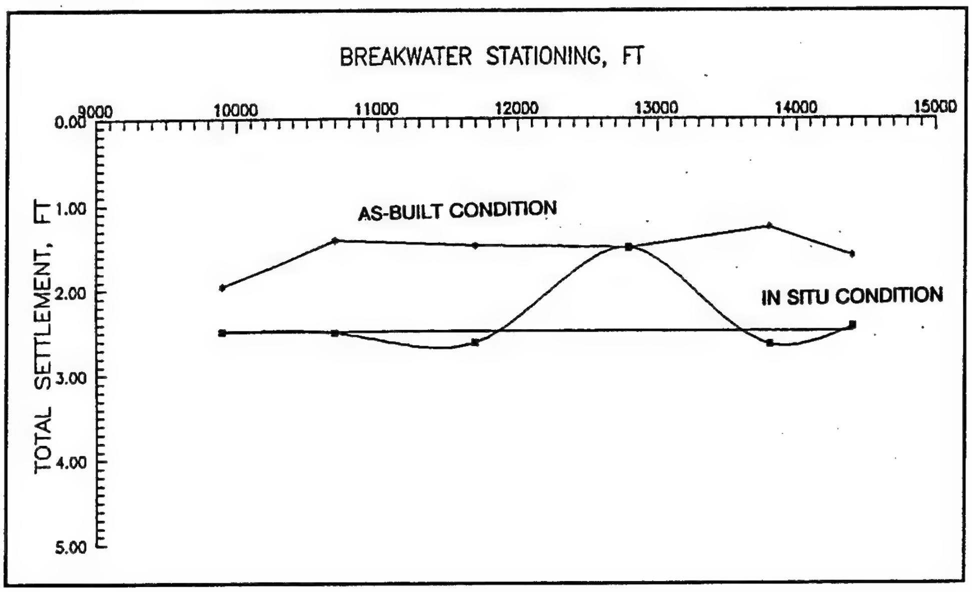


Figure 4-14. Predicted foundation settlement

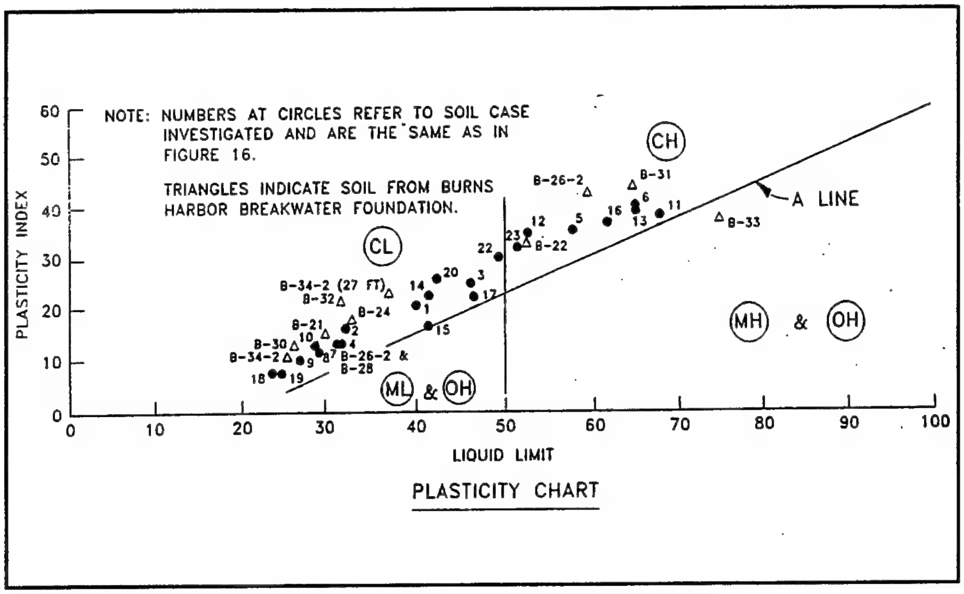


Figure 4-15. Plasticity of remolded clays from fluvial deposits (Peters et al. 1996)

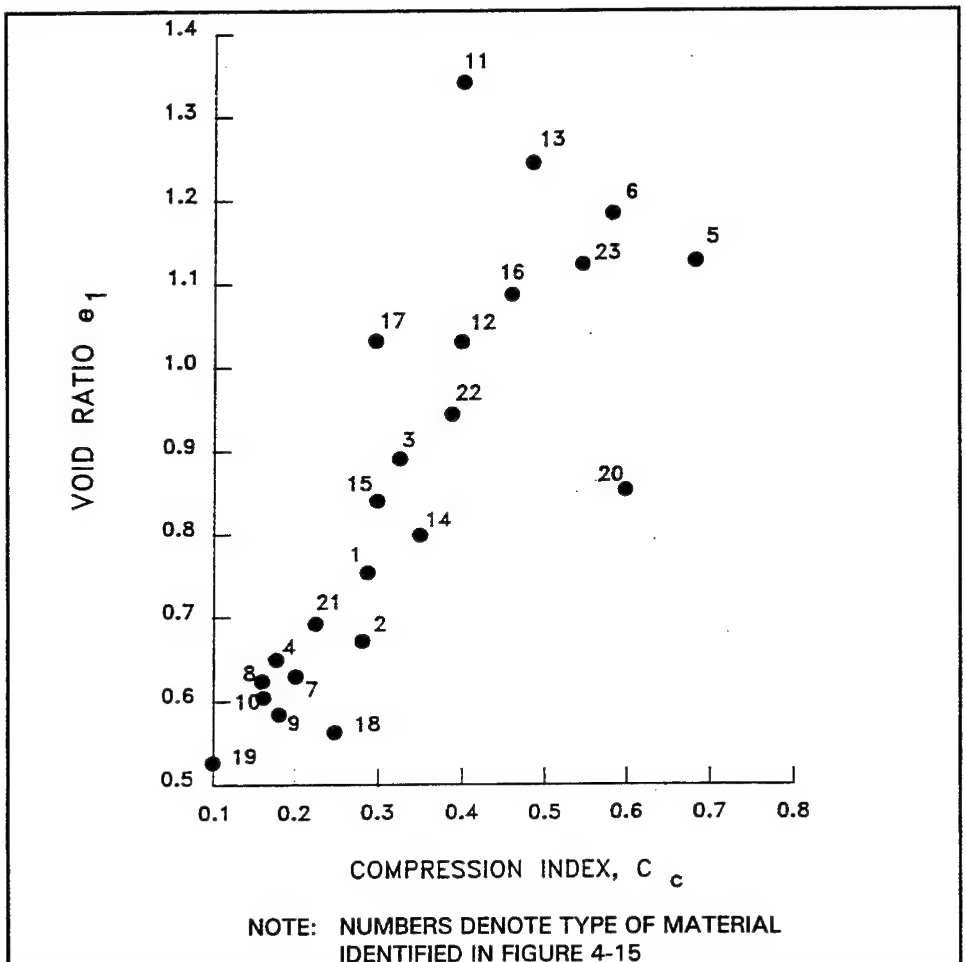
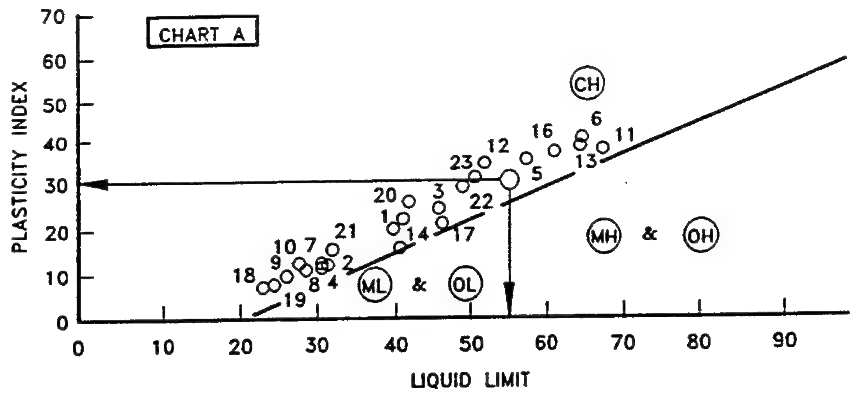


Figure 4-16. Compression indices for remolded clays versus void ratios at 1 tsf consolidation pressure

STEP 1: DETERMINE VALUES FOR LL, PI, AND INITIAL VOID RATIO FOR APPROPRIATE LOCATION AND DEPTH OF CLAY

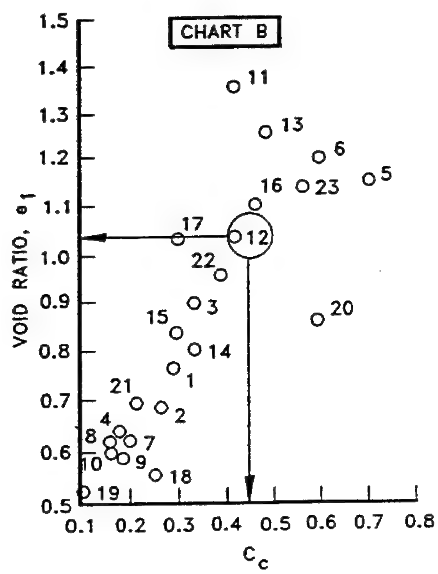
STEP 2: ENTER IN PLASTICITY CHART A AND LOCATE NEAREST POINT



PLASTICITY CHART

PLASTICITY OF REMOLDED CLAYS FROM FLUVIAL DEPOSITS

THE NEAREST POINTS TO QUALIFY = 23, 12, 5; USE 12



COMPRESSION INDICES FOR REMOLDED CLAYS

VOID RATIOS σ'_v =1 TSF

STEP 3: FIND C_c AND e_1 FOR CORRESPONDING POINT "12"

Figure 4-17. General procedure for derivation of consolidation parameters of undisturbed clay (Peters et al. 1996)

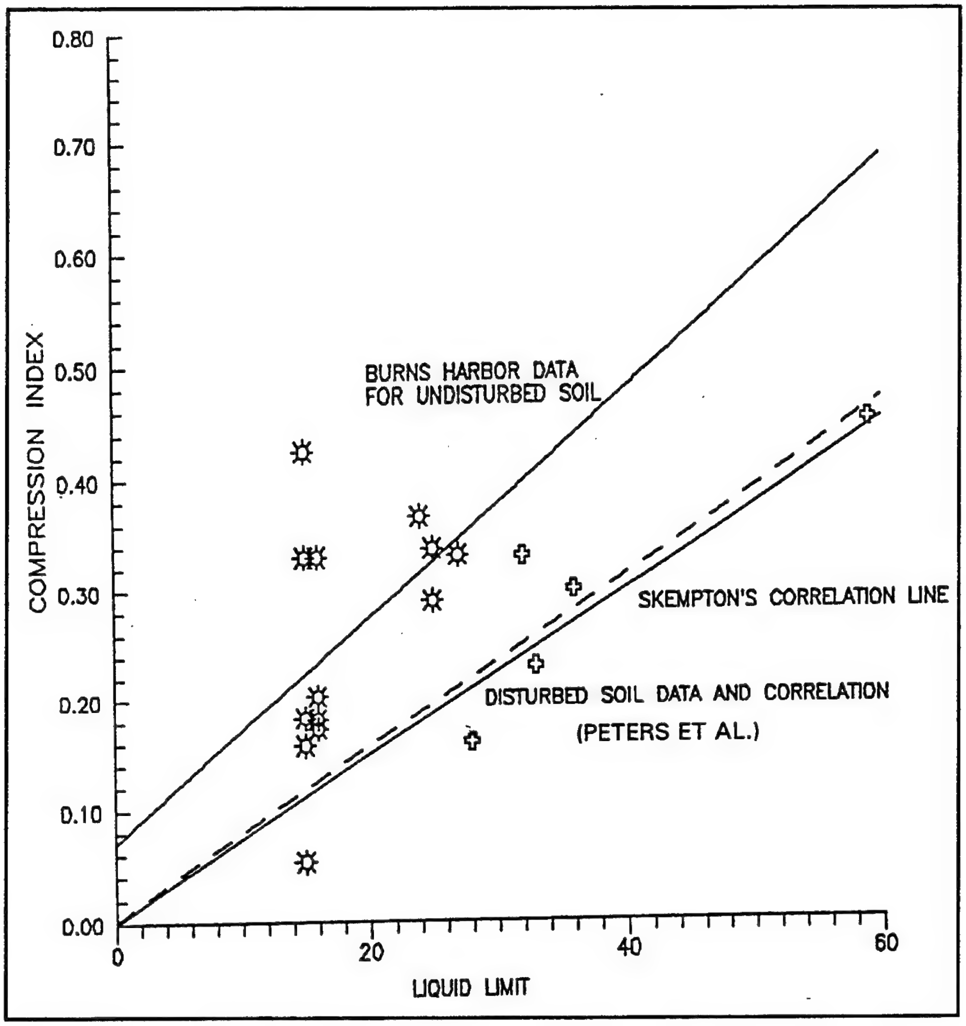


Figure 4-18. Comparison of Skempton (1944) and project soil data

fluid squeezing out of the soil skeleton. The third portion is a process of secondary consolidation and is defined as creep settlement, which reflects the viscoelastic properties of the soil. Creep settlement is much slower and lasts an indefinite period of time. The consolidation and viscoelastic settlement rates for clayey sands are in the approximate ratios of 1 to 50 based on EM 1110-2-1904 (HQUSACE 1990). The significant effects of creep settlement can be observed over a large number of years and are calculated from:

$$\rho_{creep} = [H\ C_\alpha/(1 + e_o)]\ \log(t/t_p) \tag{4-1}$$

where t is time in years and t_p is 0.5 year as suggested by Leonards, H is thickness of the layer, e_o (in situ void ratio) is taken from Table 4-1, and average C_α is assumed to be 0.003. For a 20-year period, creep settlement in the Burns Harbor area would range from 1 to 5 in. (2.54 to 12.70 cm), based on Equation 4-1.

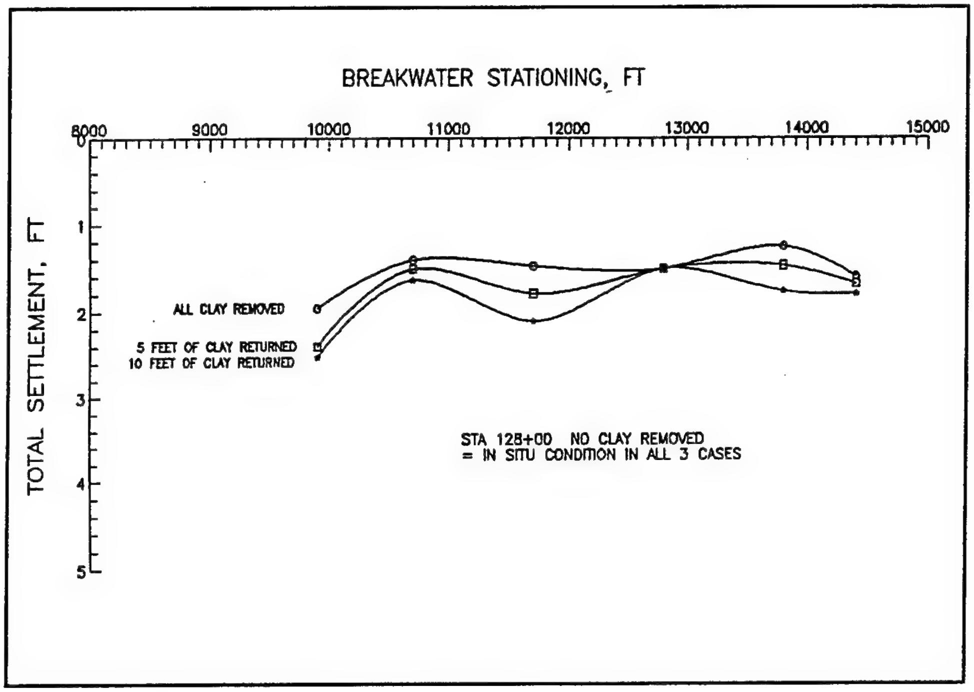


Figure 4-19. Computed settlement for possibility that clay was washed back into foundation excavation prior to sand backfilling (two cases considered)

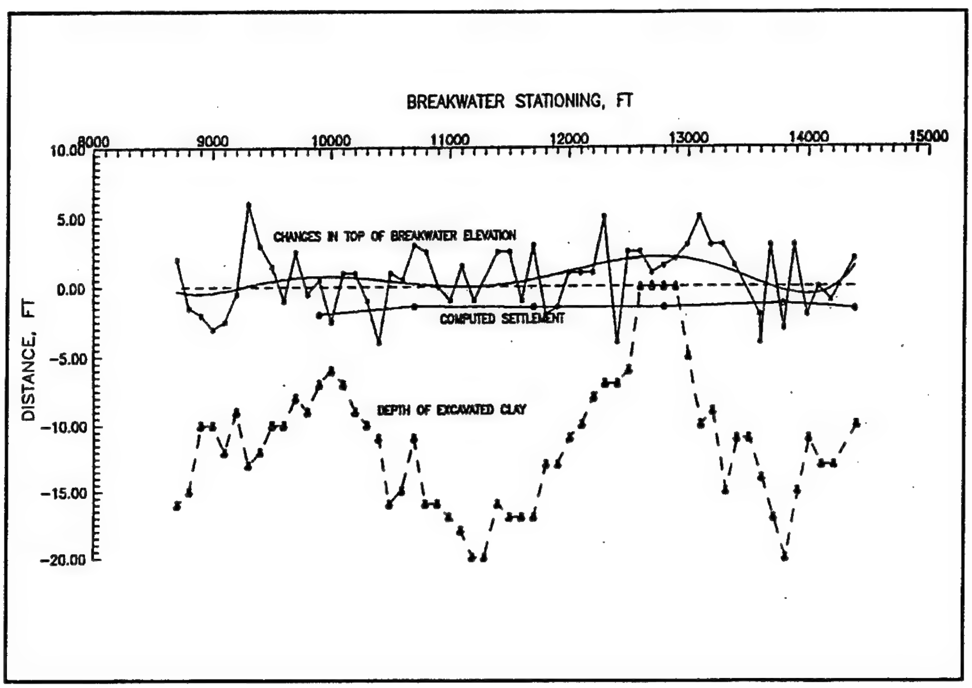


Figure 4-20. Comparison of observed and computed data

A monolithic structure can be easily surveyed with conclusive judgement as to the stability and integrity of the structure. Contrary to this, a breakwater consists of many heavy stone blocks, initially arranged in no consistent order, that are exposed to heavy wave action. The rearrangement of the structure and resulting variance between surveys do not necessarily indicate that the structure has undergone structural distress such as excessive settlement or that the supporting system was compromised. The structure was surveyed on several occasions as a part of assessment of breakwater damage.

Examination of survey cross sections taken along the breakwater during the first 10 years following construction indicates no discernible or consistent trend in the change of slope or top elevation of the breakwater as observed from Figures 4-21a through 4-21f. The top elevation of the breakwater has subsided a maximum of about 4 ft (1.2 m) in only one or two locations throughout the first 10 years of existence. In other areas, the top elevation of the breakwater has increased probably because of rearrangement of stone blocks by wave action. In summary, the survey data do not appear to reflect a settlement pattern exceeding the order of magnitude predicted by the cases evaluated in this study.

Table 4-3
Parameters for Estimating Disturbed Consolidation Properties

Sample Boring	Elevation ft.	Undisturbed Compression Index	Liquid Limit	Compression Index (Skempton)	Compression Index (Peters)
B24	520.2	.182	33	.162	.23
B26-2	536.6	.366	59	.343	.45
B28	520.65	.158	28	.12	.16
B30	508.15	.183	26	.112	
B32	528.5	.053	32	.154	.23
B34-2	565.4	.183	36	.182	.3

$Cc = 0.007\ (LL - 10)$

Conclusions

Original calculations of settlement of the Burns Harbor, Indiana breakwater structure were confirmed. Settlement during and shortly after construction probably occurred essentially as expected. This settlement is therefore not apparent in post-construction surveys. Major settlements of the breakwater crest indicated by the 1975 survey are less than 5 ft (1.5 m). The magnitude of the calculated settlements, combined with examination of both as-built cross-sectional surveys of the breakwater and annual cross-sectional surveys taken in 1975, indicate that major rearrangements of the breakwater between construction and 1975 were not due solely to settlement. No effort was made to calculate observed settlement after 1975, when significant amounts of repair stone were added to the structure. It is unknown that settlement has played any significant role in the unsatisfactory performance of the breakwater.

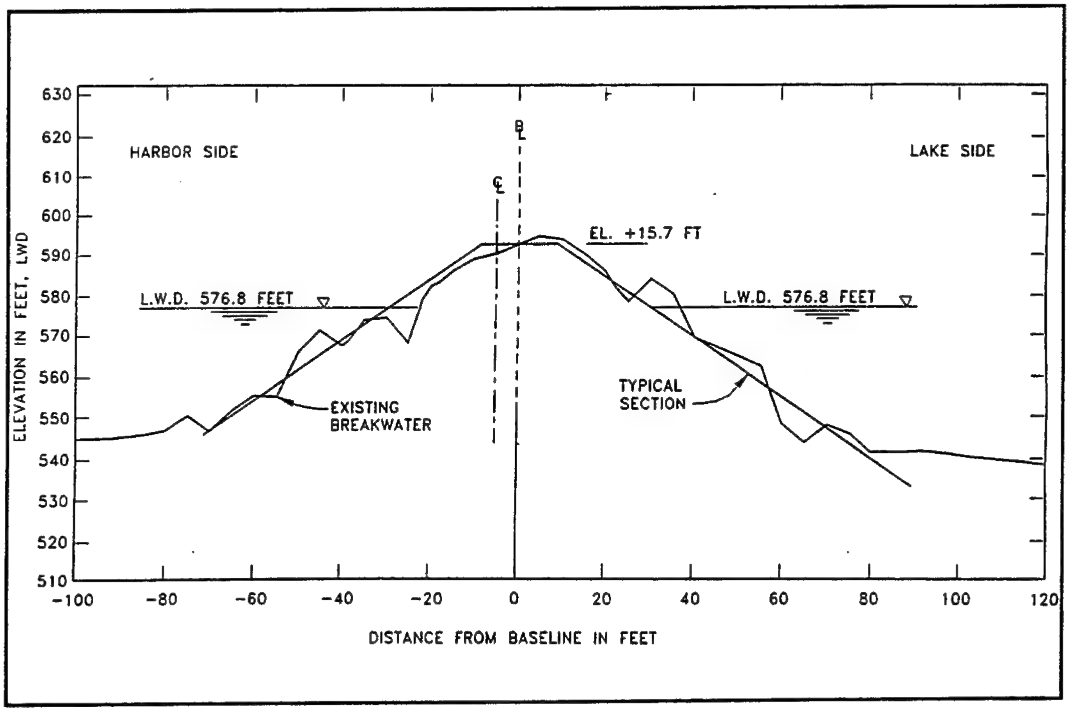


Figure 4-21a. 1985 Survey cross section, Station 23+00

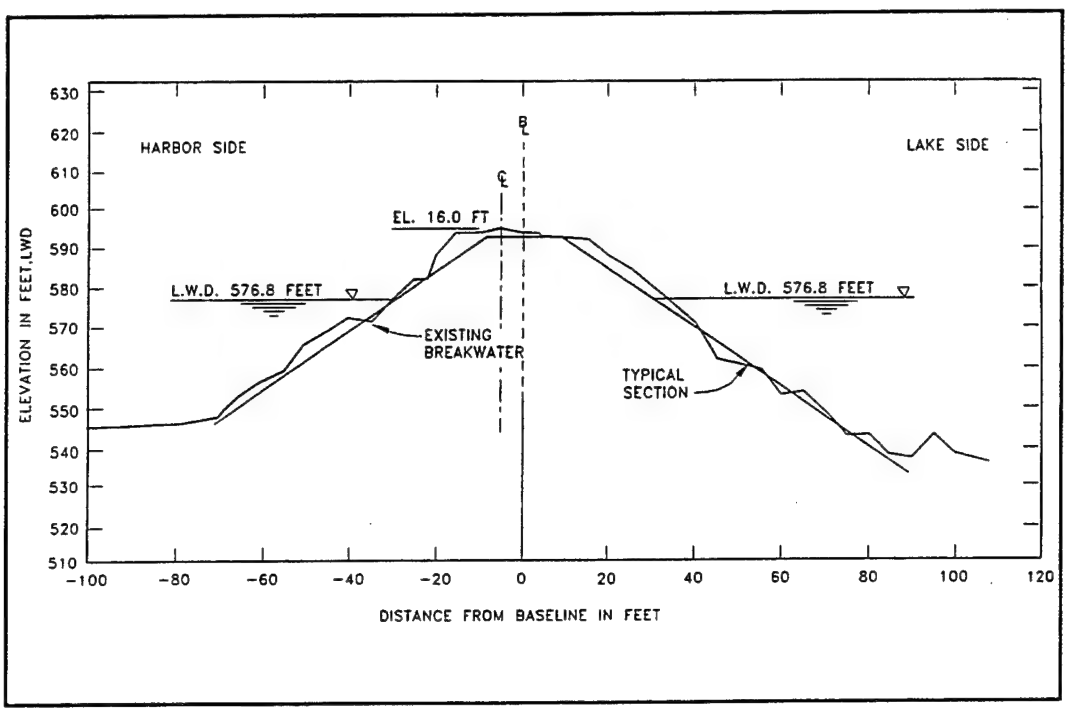


Figure 4-21b. 1985 Survey cross section, Station 24+00

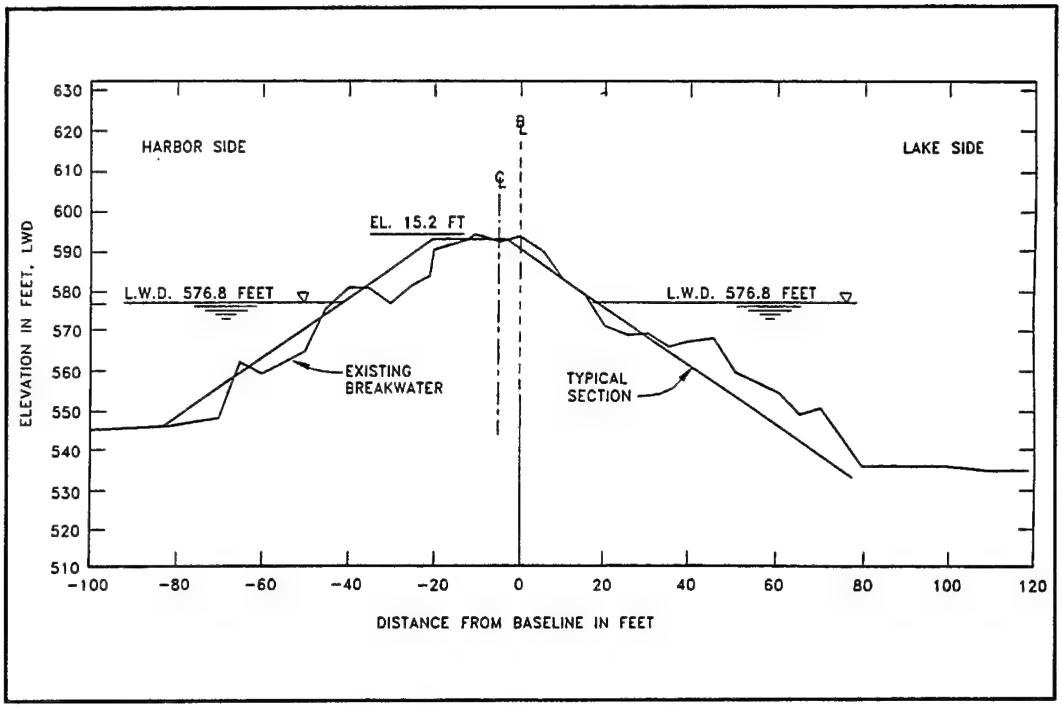


Figure 4-21c. 1985 Survey cross section, Station 26+00

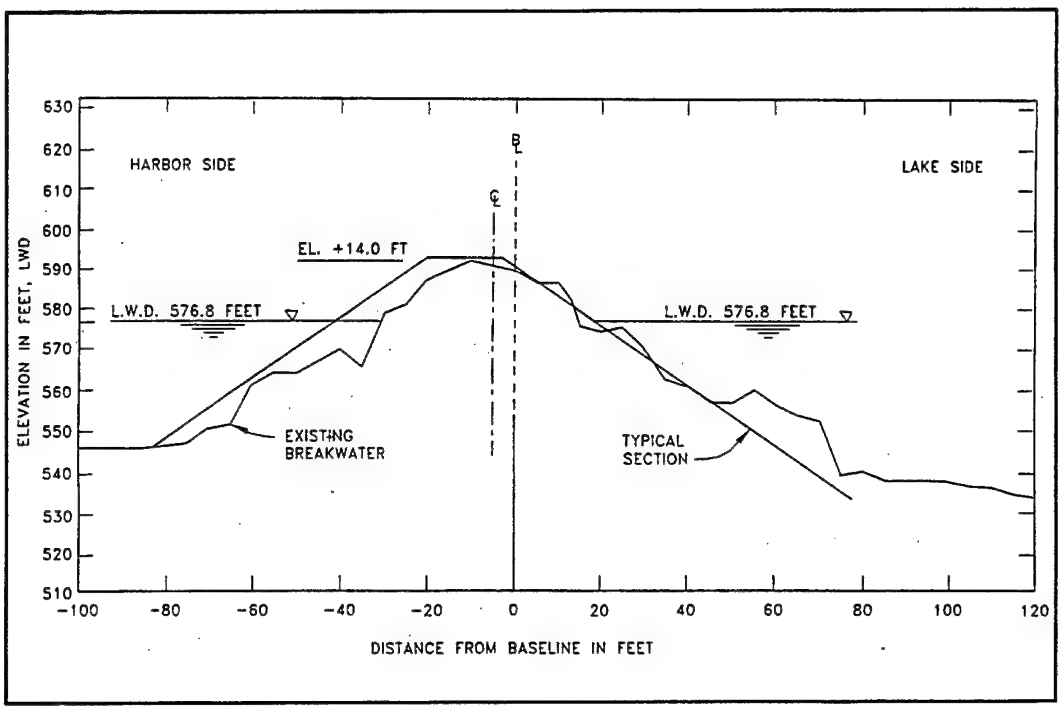


Figure 4-21d. 1985 Survey cross section, Station 29+00

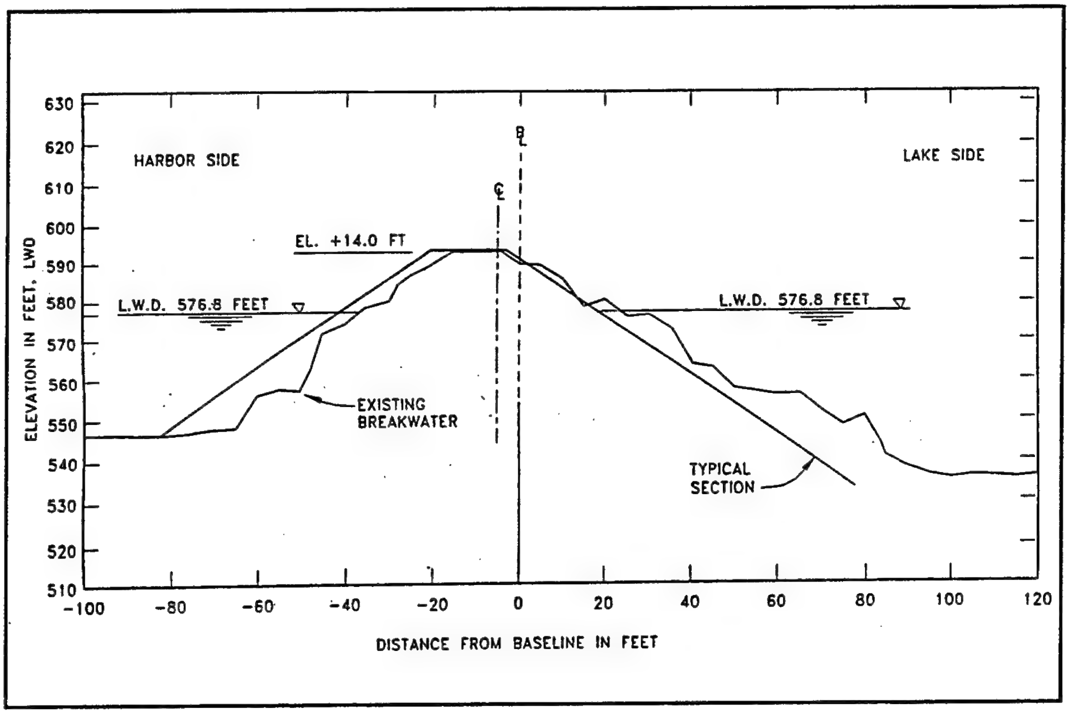


Figure 4-21e. 1985 Survey cross section, Station 32+00

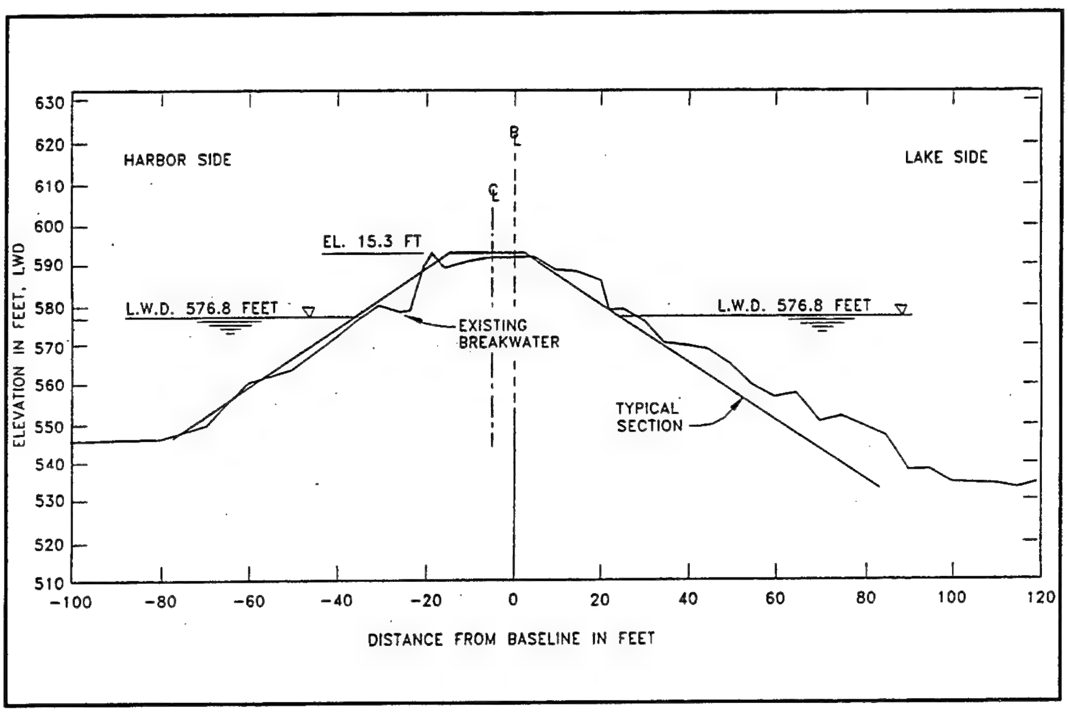


Figure 4-21f. 1985 Survey cross section, Station 34+00

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Appendix 4A
Computation of Settlement of Disturbed Clay

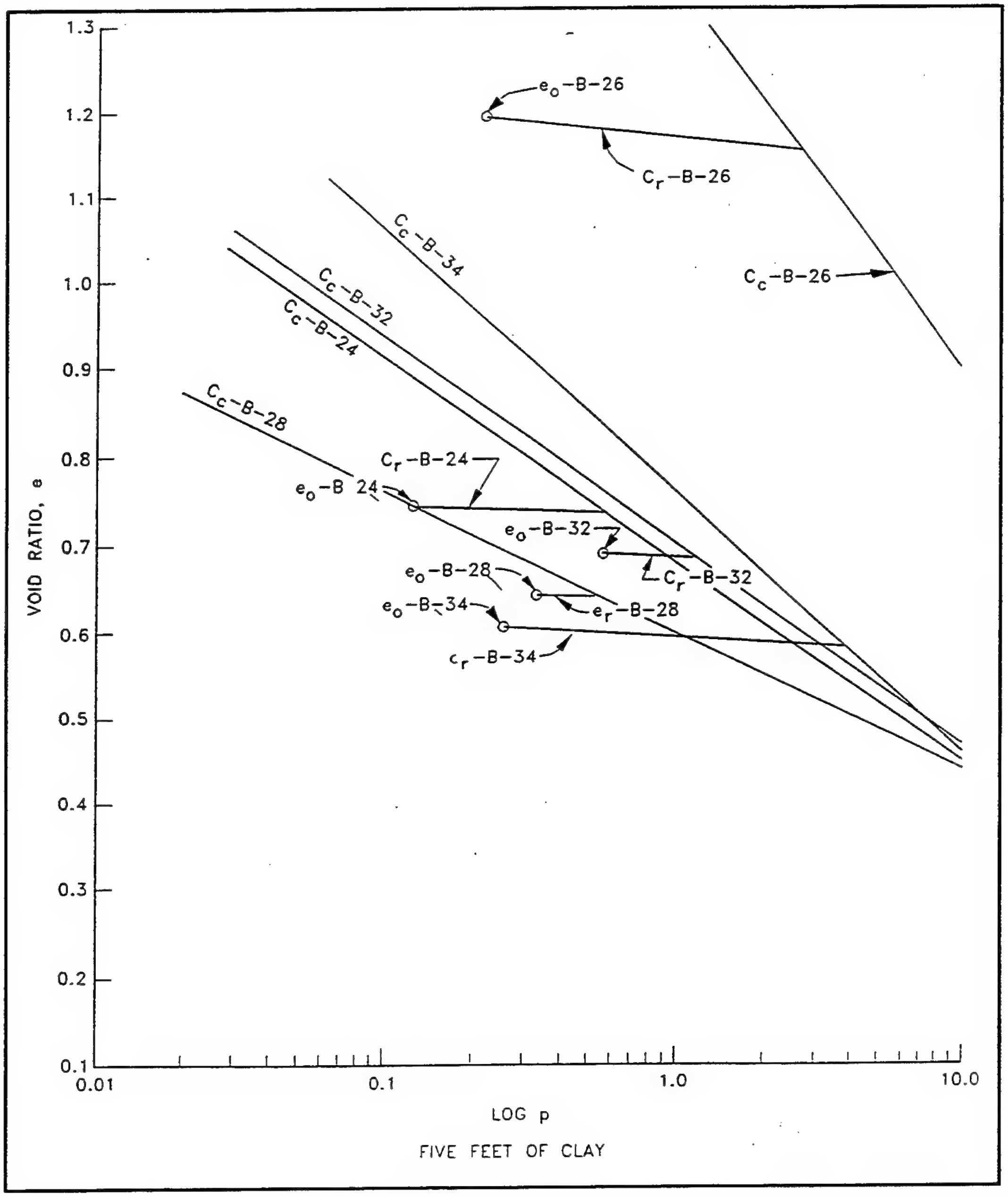

Figure 4A-1. E - logp curves

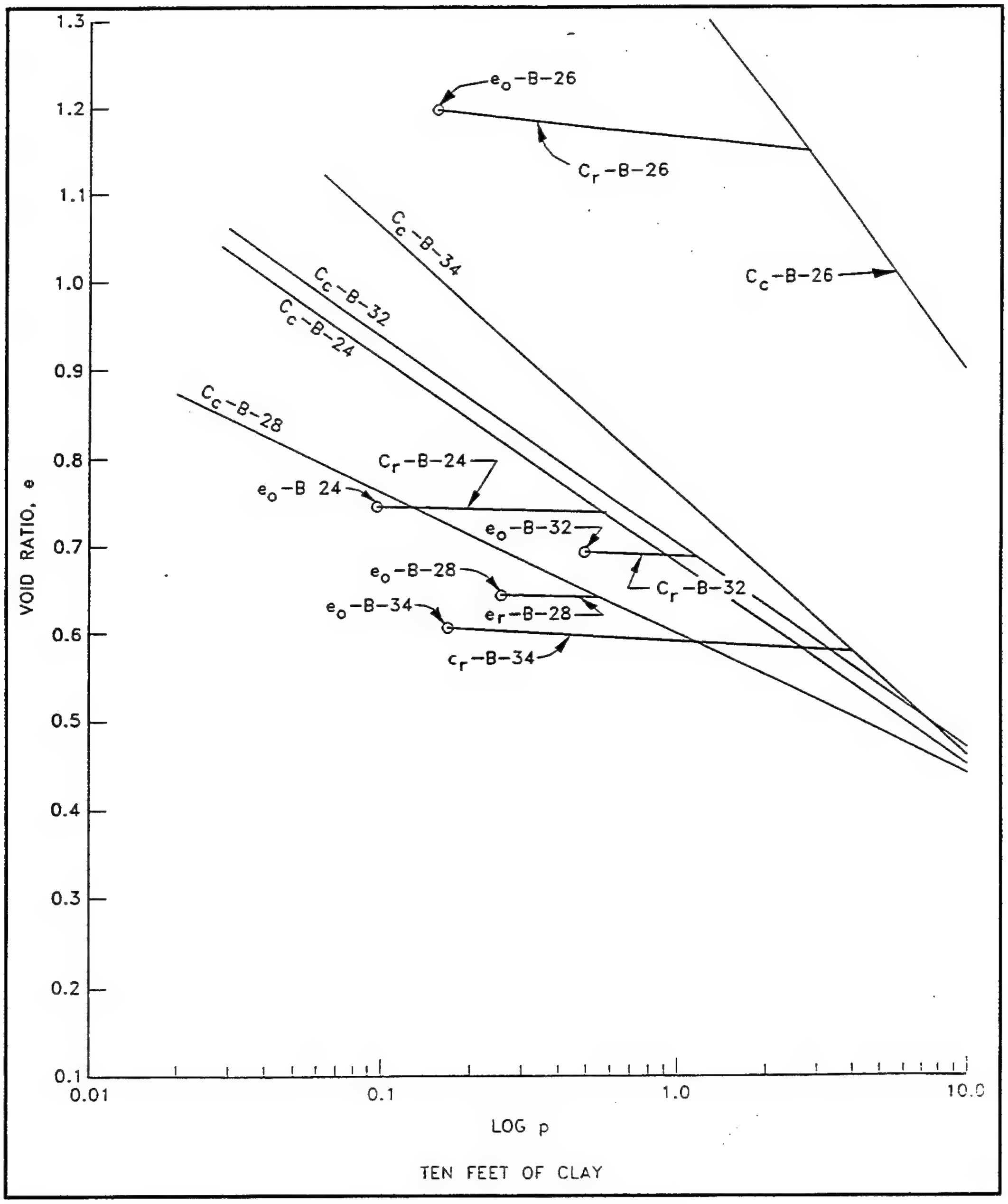

Figure 4A-2. E - logp curves

Computation of settlement of disturbed clay

B-24

$$\rho = HC_c \frac{1}{1 + e_o} \log \frac{p + \Delta p}{p}$$

$$\rho_1 = (5 \text{ ft})(0.023) \frac{1}{1 + 0.742} \log \frac{0.530}{0.125} = 0.044 \text{ ft}$$

$$\rho_2 = 5(0.23) \frac{1}{1 + 0.738} \log \frac{2.056}{0.53} = 0.389 \text{ ft} \quad 0.433\text{ft}$$

B-26

$$\rho_1 = (5)(0.045) \frac{1}{(1 + 1.195)} \log \frac{2.141}{0.233} = 0.10 \text{ ft}$$

$$\rho_2 = 0$$

B-28

$$\rho_1 = (5)(0.016) \frac{1}{1 + 0.642} \log \frac{0.53}{0.339} = 0.0091 \text{ ft}$$

$$\rho_2 = (5)(0.16) \frac{1}{1 + 0.639} \log \frac{2.203}{0.52} = 0.306 \text{ ft} \quad 0.315 \text{ ft}$$

B-32

$$\rho_1 = (5)(0.023)\,\frac{1}{1 + 0.69}\,\log\frac{1.16}{0.96} = 0.0056 \text{ ft}$$

$$\rho_2 = (5)(0.23)\,\frac{1}{1 + 0.677}\,\log\frac{2.366}{1.16} = 0.212 \text{ ft} \quad 0.217 \text{ ft}$$

B-34

$$\rho_1 = (5)(0.03)\,\frac{1}{1 + 0.606}\,\log\frac{2.148}{0.257} = 0.086 \text{ ft}$$

Table 4A-1
5- or 10-ft Clay Input

Section Boring	Settlement as Built ft	Additional Settlement 5 ft Clay	Additional Settlement 10 ft clay	Total Settlement 5 ft	Total Settlement 10 ft
B-24	1.962	0.044	0.433	2.006	2.395
B-26	1.410	0.100	0.100	1.510	1.510
B-28	1.476	0.009	0.315	1.485	1.791
B-30	in situ	in situ	in situ	N/A	N/A
B-32	1.259	0.006	0.217	1.265	1.476
B-34	1.600	0.086	0.086	1.686	1.686

5 Structural Stability Analysis

by Heidi P. Moritz[1] and Hans R. Moritz[2]

Introduction

The determination of the stability of the breakwater as a rubble-mound structure has been one of the main goals of the Burns Harbor Monitoring of Completed Coastal Projects (MCCP) Program. Several aspect of that determination are of interest including: documentation of prototype stability results for rectangular armor units, evaluation of theoretical versus actual damages, evaluation of the physical model used for the design, comparison of the maintenance frequency and practices, and predictions of future conditions for the breakwater.

The breakwater has experienced significant damage over its 20-year life, both in terms of frequency as well as magnitude. A total of 145,000 tons (132,000 mt) of armor stone has been placed on the breakwater during the period 1975 through 1989. Harborside damages occurred on the same order of magnitude as the lakeside damages.

The stone stability analysis of the Burns Harbor breakwater was accomplished using several sources of information and reanalysis techniques. The study was initiated by re-examining the original design of the breakwater. The maintenance history and the structure surveys provided documentation of location and extent of damages. Environmental forces impacting the structure since construction were determined through an investigation into specific storm events and storm water levels.

Two modes of potential damage, waves and settlement, were evaluated using state-of-the-art design theory to test current design methodology. A mass balance analysis utilized structure surveys, maintenance activities, and theoretical predictions of damages to simulate rubble-mound breakwater

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[2] Civil Engineer, U.S. Army Engineer District, Portland, OR.

based on structure orientation and degree of historical maintenance. A damage magnitude was defined that uses total maintenance and present breakwater conditions to evaluate segment vulnerability.

Breakwater Cross-Section Analysis

This chapter investigates change that occurred in the cross-sectional area of the rubble-mound breakwater protecting Burns Harbor between 1967 and 1989. Structure surveys conducted in 1967, 1975, and 1989(92) were compared in order to determine the amount and location of cross-sectional change which occurred between the three surveys. Plots of the survey profiles used for the comparison are provided in Appendix 5A.

Partial structure surveys conducted in 1985 and 1992 were used to perform consistency verification for the 1989 survey. The survey conducted in 1992 along the west arm of the breakwater was used in place of the 1989 survey for that reach since it more accurately portrays the current condition of the breakwater. The breakwater's profile between the survey years of 1967, 1975, and 1989(92) was compared to the volume (location) of maintenance stone placed on the breakwater.

Conclusions reached in this analysis are based primarily upon two time-varying statistics describing the breakwater cross section. The mean cross-sectional area of breakwater was used to determine the relative change of the breakwater in terms of stone quantity. The variance of breakwater cross sections was utilized to gauge the uniformity of the breakwater profile with respect to the structure's design template. These statistics are used to describe and quantify the breakwater's configuration throughout its surveyed history. Tests of statistical significance indicate the positions along the breakwater where the worst damage was sustained for 1975 and 1989(92).

Structure surveys for Burns Harbor rubble-mound breakwater

The 5,900-ft (1,798-m) rubble-mound breakwater was surveyed in its entirety immediately after project completion (1967 as-built survey), and in 1975. Partial surveys of the breakwater were conducted in 1985, 1989, and 1992. Each survey was conducted at 100-ft (30.5-m) (or less) intervals along the centerline/baseline of the structure, with survey lines extending perpendicularly from the breakwater center line (crest) down along the face of the breakwater, to the lake bed on each side of the structure.

Structure surveys for 1975 and 1985 were conducted prior to maintenance stone placement for the respective survey year. The 1989 surveys were conducted after maintenance stone placement for that year, but did not include

the portion of the breakwater which was maintained (Station 0+00 to 6+50[1]). For this portion of the breakwater, the stone maintenance as-built drawing was used for survey information. The 1992 survey was conducted in preparation for maintenance activities along the west arm of the breakwater (Station 46+00 to 58+00).

1967 (as-built) survey. The as-built survey of the finished breakwater surface was performed along the entire length of the breakwater (Stations 0+00 to 59+00). The methods used cannot be determined at this time. The survey was made at 100-ft (30.5-m) intervals along the center line of the structure by Peter Kiewit & Sons, the contractor responsible for breakwater construction. The survey interval along each cross section appeared to vary. Within the vicinity of the breakwater crest (≈ 17 ft (5 m) wide), the survey interval was approximately 10 ft (3 m). Beyond the crest, the survey interval appears to be inconsistent and varies from 10 to 40 ft (3 to 12 m). Survey elevations shown in the as-built drawings depict locations only where the structure slope of the breakwater exhibits a transition. It was assumed that the structure slope breaks shown on the as-built drawings represent those points actually measured during the survey.

The as-built survey (drawings) depicted a breakwater which followed the structure design neat-line very closely. There were some deviations of the breakwater cross section beyond (+) the neat line, but very few instances where the cross section fell below (-) the neat line tolerance of 1 ft (0.3 m). Overall, the 1967 as-built survey shows that the constructed profile of the breakwater conformed to specified design tolerances. The constructed profile appeared uncharacteristically smooth: the breakwater exhibited no local variations in elevation (voids or prominences).

1975 survey. This survey was performed along the entire length of the breakwater using rod/transit for the above-water (dry) portion of the breakwater and echo sounder (Bludworth ES130 with 8-deg beam width) for the below waterline (wet) portion of the breakwater. It is unknown at which point along the breakwater the rod/transit survey method ended and the fathometer-based sounding began. The recorded survey interval along each cross section, above and below the waterline, was 5 ft (1.5 m).

Cross sections were obtained at 100-ft (30.5-m) intervals along the "dry" portions of the breakwater. Soundings were obtained at 25-ft (7.6-m) intervals along the "wet" portions of the breakwater between cross-section stations. Local variations in elevation along the breakwater side slopes were recorded for both the "dry" and "wet" portions of the breakwater.

This type of detailed survey method (recording all significant elevation variations) best represents the actual structural condition of the breakwater at the time of survey. The survey was performed by the U.S. Army Engineer District, Chicago, Kewaunee field office during April 1975.

[1] Stations in ft.

1985 survey. The breakwater survey for 1985 was conducted from Stations 0+00 to 15+00 and 23+00 to 34+00 using a rod/transit for the dry portion of the breakwater and a sounding basket for the wet portion of the breakwater. The survey was conducted at 100-ft (30.5-m) intervals along the structure centerline. The survey interval was 5 ft (1.5 m) between elevations/soundings. Local variations of the breakwater surface were reported in terms of the recorded survey/sounding data. This survey was performed by the U.S. Army Engineer District, Detroit, Grand Haven Area Office during Summer 1985.

1989 survey. This survey was performed from station 7+00 to 58+00 using rod/transit for the dry portion of the breakwater and echosounder (Bludworth ES130 with 8-deg beam width) for the wet portion of the breakwater. Specifically, rod-based portions of the survey extended from breakwater crest to approximately 5 ft (1.5 m) below waterline. Fathometer-based surveys extended from 5 ft (1.5 m) below waterline to lake bed elevation. The fathometer and rod/transit survey methods overlapped at one data point (5 ft (1.5 m) below the waterline) on each transect from the breakwater crest. Survey interval along each cross section, above and below waterline, was 5 ft (1.5 m).

Cross sections were obtained at 50-ft (15-m) intervals along the breakwater center line. Local variations in elevation (prominences or voids), with respect to each cross section, were omitted during the actual survey in favor of depicting each cross section in an average sense. Elevations were "shot" in such a way as to smooth-out local variations of breakwater relief. This "culling" of extremal surface variations by the survey crew produced a smoothed representation of each breakwater cross section.

In some instances, "dry" elevation data (1 to 3 data points spanning 5 to 15 ft (1.5 to 5 m) horizontally and as much as 10 ft (3 m) vertically) on both sides of the breakwater were not recorded by the survey crew due to slippery/hazardous conditions above and along the waterline of the breakwater. Most armor stone movement occurs in this region. Exclusion of elevation data from the active portion of the breakwater makes comparison of the 1989 survey to previous surveys problematic. The 1989 riprap survey was conducted by the Detroit District, Saginaw area office during June 1989.

The 1989 survey was corrected to account for apparent fathometer-induced/smoothing bias. The bias correction was developed within this analysis and is described later in this chapter.

1989 as-built survey. An as-built survey was performed for Stations 0+00 to 6+50, by Gillen Marine, Inc., after completion of breakwater repair. The survey was performed along 25-ft (7.6-m) stations using rod-transit and sounding basket.

1992 survey. The breakwater survey for 1992 was conducted from Stations 46+00 to 58+00 using a rod/transit for the dry portion of the

breakwater and sounding basket for the wet portion of the breakwater. The survey was conducted at 50-ft (15-m) intervals along the structure center line. The survey interval was 5 ft (1.5 m) between elevations/soundings. Local variations of the breakwater surface were reported in terms of the recorded survey/ sounding data. This survey was performed by Kenneth Balk & Associates during the summer of 1992.

Combining the 1989 and 1992 surveys. Documentation of the present condition of the breakwater was performed using three survey sets due to partial coverage of each survey year. For simplicity and completeness, the 1989 as-built and 1989 structure surveys were combined into one data set, allowing the entire length of the riprap breakwater to be characterized by one common survey. The 1992 survey data for Stations 46+00 to 58+00 were used in place of the 1989 data to better characterize the present breakwater condition. This combined survey will be referred to as the 1989(92) survey.

Cross-section analysis

It was anticipated that a composite analysis of the entire 5,900-ft (1,798-m) breakwater would not provide sufficient insight into the correlation between (a) temporal/spatial breakwater damage trends, and (b) maintenance stone placement activities. Breakwater maintenance was incremental, with stone placed at many locations between 1975 and 1989. In addition, the north arm is exposed to bigger waves than the west arm.

Partitioning of the breakwater. The rubble-mound portion of Burns Harbor breakwater was subdivided into eight segments, covering Stations 0+00 to 57+00. This partitioning allowed for the systematic verification of where and when the changes in breakwater cross section occurred. Aerial coverage of each of the eight breakwater segments is shown in Figure 5-1 and is listed below.

a. Segment 1 = 0+00 to 6+00

b. Segment 2 = 7+00 to 16+00

c. Segment 3 = 17+00 to 22+00

d. Segment 4 = 23+00 to 31+00

e. Segment 5 = 32+00 to 37+00

f. Segment 6 = 38+00 to 46+00

g. Segment 7 = 47+00 to 50+00

h. Segment 8 = 51+00 to 57+00

Breakwater stations between 57+00 and 59+00 were not included in this analysis, due to the highly variable nature of this area and its proximity to the steel sheet pile cellular breakwater.

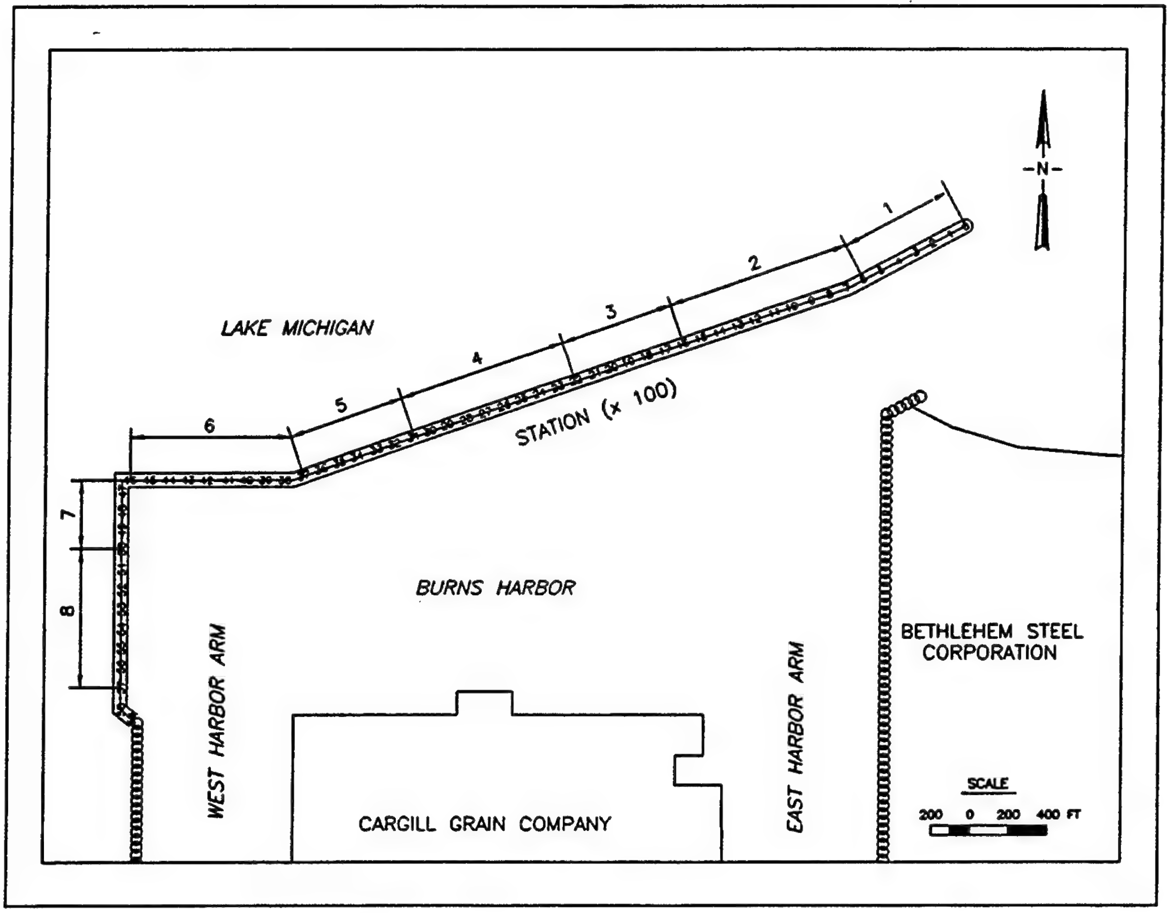

Figure 5-1. Breakwater segments

These segments were chosen according to the amount and location of maintenance stone placed on the breakwater and the spatial orientation of the breakwater arms. Each segment represents a zone where relatively equal amounts of armor stone were placed on the breakwater in a similar harborside and/or lakeside location. The location and amount of additional armor stone (maintenance) placed on the breakwater between 1975 and 1989 are shown in Figure 5-2 and Table 5-1. No maintenance stone was placed on the rubblemound breakwater between 1967 and 1975.

Previous attempts to determine breakwater cross-sectional change. A previous study of cross-sectional changes in the Burns Harbor rubble-mound breakwater was undertaken in 1975 (U.S. Army Engineer District, Chicago 1976). Results of this study were based upon horizontal control which was established along the structure in 1967 and 1975. The 1967 horizontal control was presented in terms of a structure center line. The horizontal control established in 1975 was presented in terms of a structure baseline. Later surveys also used the structure baseline. The 1975 breakwater study assumed that the baseline and center line were coincident.

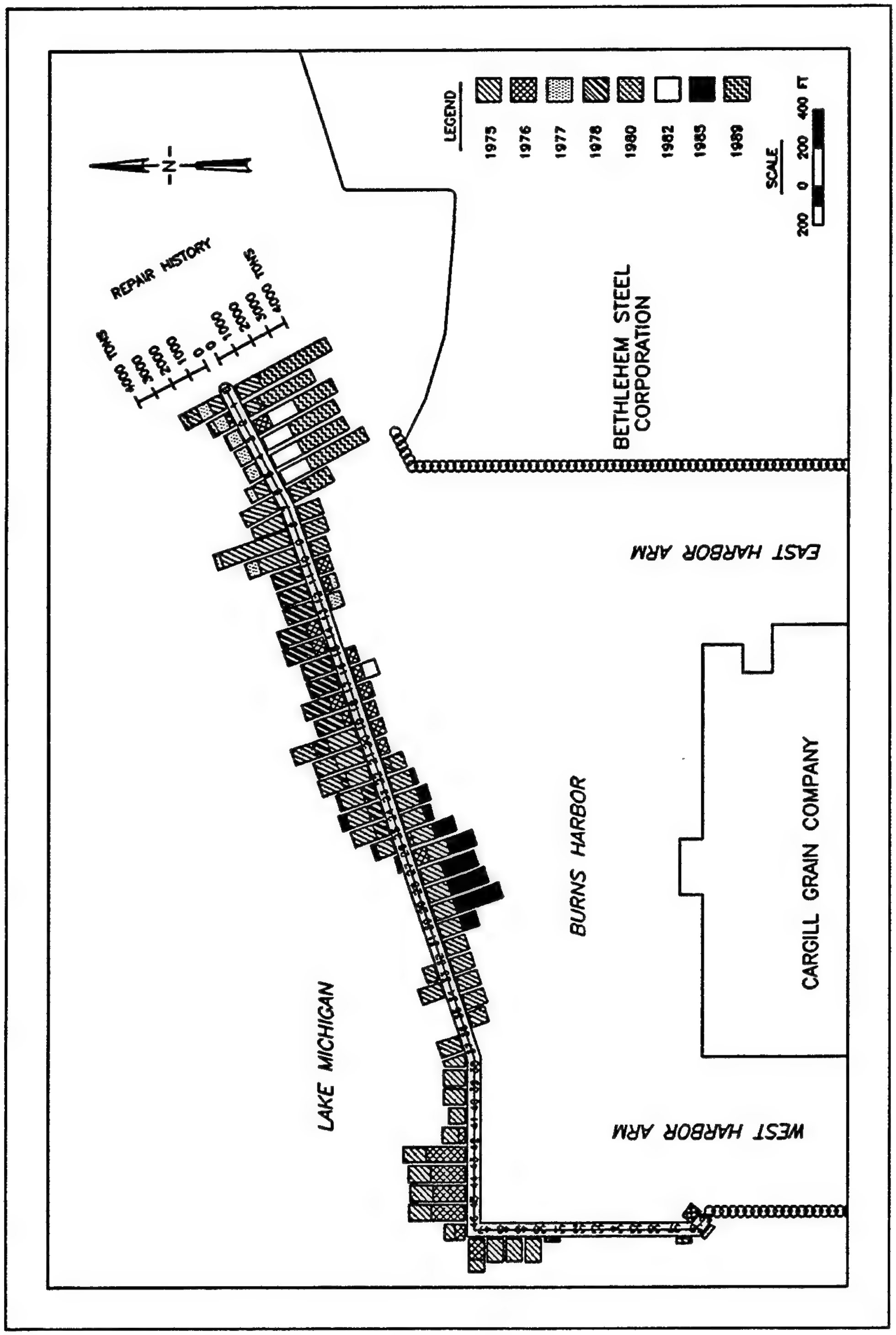

Figure 5-2. Breakwater repair history

During the present study, thorough investigation of the 1967 and 1975 horizontal control revealed that the baseline and center line did not coincide. The baseline and center line could not be located in relative position by any constant offset value.

Table 5-1
Maintenance (Armor) Stone Placement at Burns Harbor Breakwater for 1975-1989

Segment No.	Length lin ft	Actual Amount of Maintenance Stone Placed		
		Harborside tons	Lakeside tons	Total tons
1	600	27,336	7,188	34,524
2	900	5,767	18,789	24,556
3	500	3,239	12,704	15,943
4	800	21,069	5,232	26,301
5	500	4,388	2,925	7,313
6	800	0	17,585	17,585
7	300	0	3,510	3,510
8	600	0	0	0
Totals		61,799	67,933	129,732

Note: Stone amounts placed in 100-ft "buffer" zones separating breakwater segments are not included.

The above observations made the quantitative comparison of 1967, 1975, and 1989 survey data, based on horizontal control, problematic. Had consistent horizontal control existed, separate evaluation of lakeside and harborside cross-sectional change would have been possible. Considerations governing the establishment of horizontal control along the breakwater are discussed in Appendix 5B.

Vertical subdivision of individual cross sections. For this report, comparison of breakwater cross-sections was performed in terms of vertical control rather than horizontal control. The cross-sectional area of the breakwater was resolved into two vertical regions. The analyses investigated cross-sectional changes in both the upper and lower regions of the structure. However, it was not possible to specify whether those changes occurred on the harbor side or lakeside.

In summary, a vertical method of control was utilized to subdivide each survey-generated cross section into upper and lower regions. This approach was implemented in order to: (a) overcome the uncertainty associated with orienting/comparing different station cross sections using an unreliable means

of horizontal control, and (b) separate the overall cross section of the breakwater into regions governed by different cause and effect processes. The demarcation of the characteristic overall cross section into upper and lower regions is described below.

a. *Upper cross section.* Encompasses each station cross section between the elevations of +14 ft (+4.3 m) low water datum (LWD) (breakwater crest) and -10 ft (-3 m) LWD. The upper cross-sectional design template represents approximately 35 percent of the entire breakwater cross section (design template area = 1,272 sq ft (118 m^2)).

b. *Lower cross section.* Encompasses each station cross section between the elevations of -10 ft (-3 m) LWD and -30 ft (-9.1 m) LWD (breakwater base). The lower cross-sectional design template represents approximately 65 percent of the entire breakwater cross section (design template area = 2,380 sq ft (221 m^2).

c. *Overall cross section.* Encompasses each station cross section between the elevations of +14 ft (4.3 m) LWD (crest) and -30 ft (-9.1 m) LWD (base). The overall cross-sectional design template represents approximately 90-100 percent of the entire breakwater cross section (design template area = 3,652 sq ft (339 m^2).

To conduct a consistent statistical analysis of breakwater cross-sectional area, it was necessary to establish a common lower limit for area measurement which was applicable to all 57 cross sections. Due to varying lake bed elevations, a lower limit of -30 ft (-9.1 m) LWD was established. According to the 1989 survey, actual lake bed elevations vary along the lakeside of the breakwater from -30 ft to -41 ft (-9.1 m to -12.5 m) LWD, and along the harbor side of the breakwater from -26 to -35 ft (-8 to -11 m) LWD.

The overall cross section template encompasses each station cross section between the elevations of +14 ft (+4.3 m) LWD (crest) and -30 ft (-9.1 m) LWD (base). Depending upon station location, the overall cross-section template represents approximately 90 to 100 percent of the entire breakwater cross section as shown in the 1989 survey (overall design template area = 3,652 sq ft (339 m^2). It is assumed that the overall cross section will include all damages due to wave/ice activity and a majority of the damages due to settlement, if any occurred. Armor units which are displaced below -30 ft (9.1 m) LWD or outside the overall cross section do not contribute to the breakwater's function and are considered to be damaged or lost stone.

Locations of the two cross-sectional regions are shown schematically in Figure 5-3. The overall cross-sectional area at a particular breakwater survey station can be approximately determined in terms of the upper and lower cross-sectional regions as:

Overall cross-sectional area = upper region + lower region

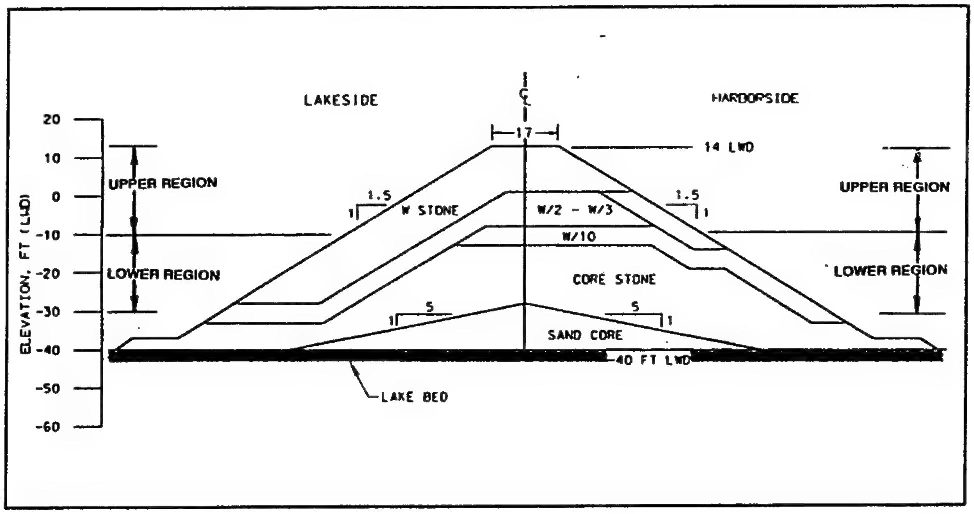

Figure 5-3. Vertical subdivision of cross section

The -10-ft (-3-m) LWD cross-sectional boundary partitioning the overall cross section of the breakwater into upper and lower regions was established using stability criteria which describe the aspects of wave-induced armor stone movement. The limit of significant wave-induced turbulence/armor stone agitation is assumed to extend downward from the still-water level for approximately one wave height. The -10-ft (-3-m) LWD elevation was determined by subtracting the original design wave height of 13 ft (4 m) from the typical instantaneous lake water level at the breakwater site. Computationally, this is expressed as:

3.5 ft LWD (5-yr water level) - 13.0 ft (2-yr wave) = -9.5 (-10.0 ft LWD)

In addition to wave-induced forces, the rubble-mound breakwater is periodically subjected to ice-induced forces. Massive ice ridges can form on the lakeside of the breakwater. The upper lakeside portion of the breakwater can be subjected to large overturning/uplift forces due to wave-induced ice flow motions impinging on the breakwater face. The upper harborside portion of the breakwater can be subjected to large downthrust forces caused by an ice sheet overtopping the breakwater crest and sliding along the breakwater slope during massive ice floes. The depth at which ice may affect the rubble-mound breakwater is estimated at 10 to 15 ft (3 to 5 m) below the water level. Although the magnitude of ice-induced forces at Burns Harbor breakwater was not quantified, experience with offshore structures in similar environments indicates that lake ice probably has an effect on the breakwater.

The combined independent effects of lake-fast ice and storm waves upon the breakwater are believed to be manifested within the upper portion of the breakwater; specifically above the -10-ft (-3-m) LWD elevation.

The upper and lower regions are subject to foundation-induced changes in cross-sectional area. Adverse foundation effects include: (a) en masse

settlement of the lake bed, causing displacement of the breakwater; and (b) differential lake bed settlement, causing local slope instability along the breakwater face.

Assessment of cross-sectional area change

The breakwater's armor stone layer above the -10-ft (-3-m) LWD elevation was assumed to represent the "active" profile of the breakwater. The "active" profile is defined as that region where most of the wave- or ice-induced movement of armor stone occurs on the breakwater. The cross-sectional region below -10 ft (-3-m) LWD was postulated as "passively" changing due to the redistribution (avalanching) of armor stone originating from the upper cross section. Both upper and lower regions are subject to settlement-induced effects of the lake bed.

If armor stone is dislodged from the breakwater by wave- or ice-induced forces, the stone can either remain within the overall cross section, or be displaced completely from the overall cross section.

En masse and local settlement would result in armor stone being displaced under the -30-ft (-9.1-m) LWD limit of the overall cross section. Thus, the affected stone would be "lost" from the overall cross section. Side slope instabilities due to local settlement would have the same effect as for wave- and ice-induced movement of stone, in terms of the final disposition of displaced stone.

Control volume approach. The above assessment of armor stone (re)distribution established the concept of a "control volume" which was applied to the overall cross section (+14 to -30 ft (4.3 to 9.1 m) LWD). Stone which avalanches from the upper region onto the lower region remains within the overall breakwater cross section, resulting in no net loss of stone. Stone which is displaced completely outside the overall cross section is "lost" from the control volume. Thus, the lower region of the breakwater could increase in cross-sectional area at the expense of the upper region, even though no stone had been placed on the breakwater. The converse would not be valid: Stone would be incapable of migrating up the breakwater cross section. Should a breakwater segment experience a net <u>loss</u> in overall cross-sectional area, two explanations could be advanced to explain the occurrence:

a. Armor stone was toppled completely off the overall breakwater region onto the adjacent lake bed. This process would be depicted by stray armor stones distributed at or away from the normal rock line of the structure.

b. The breakwater segment underwent some form of foundation failure. This process would be portrayed by the overall cross section of a particular segment remaining within the design template (normal rock line boundaries) while the cross-sectional area decreased with time.

If no maintenance stone was placed on a given breakwater location during the span of two different surveys, there should be no net increase in overall cross-sectional area. No maintenance accompanied with no net loss of overall cross-sectional area could occur only when there is no significant settlement <u>and</u> stone displaced by wave/ice activity falls onto the lower region, remaining within the control volume.

Assessing the 1989 surveys using the control volume approach. In some cases, comparison of previous surveys to the 1989 survey indicated that the overall cross section had gained in area, despite no additional armor stone being placed on the breakwater. The 1989 cross sections appeared to be "swelled" in terms of the below-waterline profile (Figure 5-4). Comparing the 1985 and 1989 surveys in terms of individual cross sections within Segment 2, shows that the 1989 survey exhibited a bias on both sides of the breakwater. No stone was placed on Segment 2 between 1985 and 1989. Clearly, Segment 2 should not have increased in size (area) if no stone was placed on this part of the breakwater.

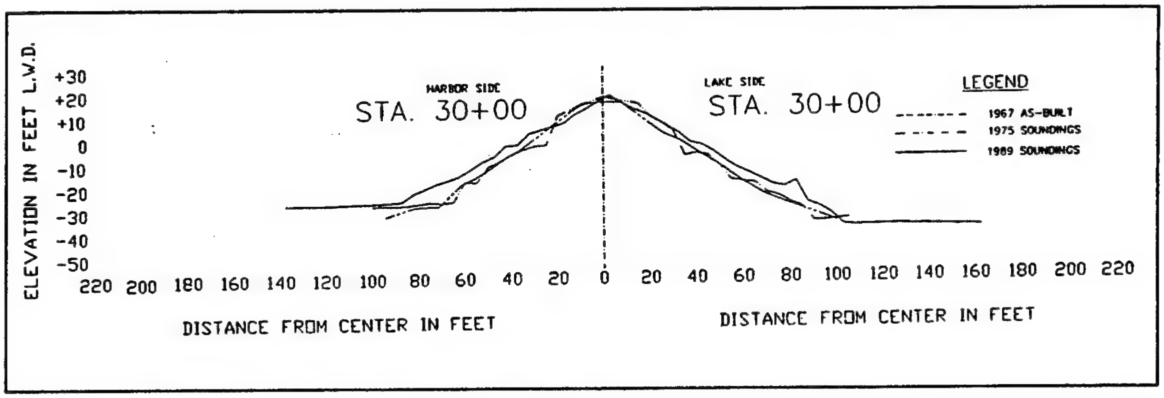

Figure 5-4. 1989 breakwater profile showing distortion below waterline

The 1985 surveys were conducted using a sounding basket and exhibited a much better indication of actual breakwater slope relief. The 1989 structure surveys were conducted using a fathometer (Bludworth ES130) and produced results which exhibited a relatively smooth profile for each breakwater cross section. Incidentally, the same type of fathometer was used for the 1975 structure survey which produced a much better (irregular) resolution of the breakwater profile than did the 1989 survey. When the 1975 survey was compared to that of 1967, no observable trends of bias were detected. The Bludworth ES130 fathometer was considered state of the art in 1975. Use of 15-year-old marine electronics in 1989 may have been the source of the bias for that particular survey.

Reduction of bias in the 1989 surveys. Given that the 1989 survey included a degree of bias, a method was developed to remove the bias from the survey data. This would allow a consistent unbiased comparison between the 1967, 1975, and 1989 surveys.

Only breakwater segments which had no maintenance stone placed between surveys could be examined in order to determine the degree of bias present in the 1989 surveys. The bias identification/calculation process was performed on breakwater Segments 2, 5, and 8. The difference between the 1985 and 1989 surveys had indicated that the overall cross-sectional template of Segment 2 had gained an average of 313 sq ft (29 sq m). This apparent increase in cross-sectional area was assumed to be equally distributed on each side of the breakwater profile.

Average biases were determined for the north arm and west arms of the breakwater. North arm bias was determined from comparisons of Segments 2 and 5 for the 1985 and 1989 surveys. West arm bias was determined by comparison of 1975 and 1989 surveys for Segment 8. The 1989 survey was corrected by subtracting the measured bias from respective breakwater segments. Table 5-2 shows the average bias which was subtracted from the 1989 survey for segments along the north and west arms of the breakwater. Information from the 1992 survey was substituted for 1989 data for Segments 7 and 8 and therefore the bias for the west arm was not used in the final calculations.

Table 5-2
Calculated Bias for the 1989 Structure Survey of Burns Harbor Breakwater Applied at Each Cross Section

Breakwater Location	Cross Section Bias, sq ft		
	Upper	Lower	Overall Total
North Arm (Sta. 7+00 - 46+00)	49	163	212
West Arm (Sta. 47+00 - 57+00)	67	30	97

Statistics utilized for cross-sectional comparison

The primary statistics utilized to compare breakwater segments were the cross-sectional area mean (x) and cross-sectional area variance (s^2). Cross-sectional mean and variance were determined for each breakwater segment and respective cross-sectional region. These statistics are shown in Table 5-3 along with other pertinent data. The statistical mean (x) was compared to the design template (μ).

The population mean (μ) for each cross section corresponds to the design template (area) for that cross-sectional region. Ideally, the variance (s^2) for the breakwater cross section for any given survey should be within the design tolerances (σ^2) as specified for construction purposes. This concept assumes that (a) the breakwater was constructed and maintained within the design tolerances, and (b) the breakwater would not experience significant

Table 5-3
Summary of Statistical Parameters for Breakwater Segments

Segment # Station	Region ft, LWD	Year	Cross-sectional Area Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	Diff. From Template ft^2	Percent Change	Number of Observations
1 (0-6)	+14 to -10	1967	1241	1190	1300	1206	19	-31	-2%	7
		1975	1226	1135	1372	7051[1]	46	-46	-3%	7
		1989	1566	1468	1623	6639	45	+294	+23%	7
	-10 to -30	1967	2586	2410	2680	7829	48	+206	+9%	7
		1975	2579	2400	2810	19986[1]	77	+199	+8%	7
		1989	2714	2624	2802	13988	65	+334	+14%	7
	+14 to -30	1967	3827	3600	3925	13407	63	+175	+5%	7
		1975	3805	3600	4110	34597	101	+153	+4%	7
		1989	4281	4157	4422	29498	94	+629	+17%	7
2 (7-16)	+14 to -10	1967	1206	1155	1240	643	11	-66	-5%	10
		1975	1124[2]	1000	1255	7218	37	-148	-12%	10

(Sheet 1 of 6)

[1] Indicates that respective cross section region is statistically more irregular than design tolerances
[2] Indicates that respective cross section region is statistically less than the design template

Table 5-3 (Continued)

Segment # Station	Region ft, LWD	Year	Cross-sectional Area					Diff. From Template		Number of Observations
			Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	ft^2	Percent Change	
		1989	1361	1169	1621	17916[1]	59	+89	+7%	10
	-10 to -30	1967	2480	2380	2545	2241	21	+100	+4%	10
		1975	2467	2317	2574	5262	32	+87	+3%	10
		1989	2488	2286	2594	9158[1]	42	+108	+5%	10
	+14 to -30	1967	3686	3595	3775	3980	28	+34	+1%	10
		1975	3591	3427	3745	11532	47	-61	-2%	10
		1989	3849	3504	4215	42052[1]	90	+197	+5%	10
3 (17-22)	+14 to -10	1967	1217	1180	1270	877	18	-55	-4%	6
		1975	1110[2]	920	1260	15302[1]	75	-162	-13%	6
		1989	1462	1375	1657	16099[1]	76	+190	+15%	6
	-10 to -30	1967	2429	2330	2565	7174	51	+49	+2%	6
		1975	2508	2385	2619	7148	51	+128	+5%	6
		1989	2532	2452	2587	2873	32	+152	+6%	6

(Sheet 2 of 6)

Table 5-3 (Continued)

Segment # Station	Region ft, LWD	Year	Cross-sectional Area					Diff. From Template		Number of Observations
			Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	ft^2	Percent Change	
	+14 to -30	1967	3646	3535	3835	11124	64	-6	0%	6
		1975	3618	3380	3826	33237	110	-34	-1%	6
		1989	3994	3799	4235	22194	90	+342	+9%	6
4 (23-31)	+14 to -10	1967	1237	1170	1310	2767	24	-35	-3%	9
		1975	1183[2]	1100	1360	6563	38	-89	-7%	9
		1989	1464	1296	1599	13316[1]	54	+192	+15%	9
	-10 to -30	1967	2446	2335	2625	9220	45	+66	+3%	9
		1975	2509	2365	2755	11734[1]	50	+129	+5%	9
		1989	2601	2432	2739	9757	46	+221	+9%	9
	+14 to -30	1967	3683	3525	3875	14475	56	+31	+1%	9
		1975	3692	3545	3985	19763	65	+40	+1%	9
		1989	4065	3805	4347	29505	80	+413	+11%	9
5 (32-37)	+14 to -10	1967	1273	1205	1320	2447	30	+1	0%	6
		1975	1266	1140	1480	19444[1]	84	-6	-1%	6

(Sheet 3 of 6)

Table 5-3 (Continued)

Segment # Station	Region ft, LWD	Year	Cross-sectional Area					Diff. From Template		Number of Observations
			Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	ft^2	Percent Change	
		1989	1230	1192	1332	3043	33	-42	-3%	6
	-10 to -30	1967	2538	2430	2620	5067	43	+158	+7%	6
		1975	2630	2465	2855	20620[1]	87	+250	+11%	6
		1989	2409	2176	2563	24834[1]	95	+29	+1%	6
	+14 to -30	1967	3812	3670	3935	7727	53	+160	+4%	6
		1975	3896	3670	4080	23994	93	+244	+7%	6
		1989	3639	3368	3895	38120	118	-13	0%	6
6 (38-46)	+14 to -10	1967	1266	1160	1470	8078	42	-6	0%	9
		1975	1281	1160	1281	9572	46	+9	1%	9
		1989	1400	1152	1696	25859[1]	80	+128	+10%	8
	-10 to -30	1967	2484	2182	2615	19799[1]	66	+104	+4%	9
		1975	2502	2091	2725	46638[1]	101	+122	+5%	9
		1989	2447	2287	2658	15610[1]	63	+67	+3%	8
	+14 to -30	1967	3749	3570	3915	11812	51	+97	+3%	9

(Sheet 4 of 6)

Table 5-3 (Continued)

Segment # Station	Region ft, LWD	Year	Cross-sectionall Area					Diff. From Template		Number of Observations
			Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	ft^2	Percent Change	
		1975	3783	3326	4160	69479[1]	123	+131	+4%	9
		1989	3846	3439	4230	68637[1]	131	+194	+5%	8
7 (47-50)	+14 to -10	1967	1294	1230	1400	6123	64	+22	+2%	4
		1975	1184[2]	935	1345	31506[1]	145	-88	-7%	4
		1992	1171[2]	1065	1325	13223[1]	94	-101	-8%	4
	-10 to -30	1967	2442	2252	2615	22202[1]	122	+62	+3%	4
		1975	2371	2245	2462	11271[1]	87	-9	0%	4
		1992	2132[2]	2056	2218	4471	55	-248	-10%	4
	+14 to -30	1967	3736	3652	3920	15438	102	+84	+2%	4
		1975	3554	3255	3800	53189[1]	189	-98	-3%	4
		1992	3246[2]	3122	3314	18321	111	-406	-11%	4
8 (51-57)	+14 to -10	1967	1271	1200	2245	3312	31	0	0%	7
		1975	1177[2]	1100	1335	7065	46	-95	-7%	7
		1992	1045[2]	983	1171	5859	42	-227	-18%	7

(Sheet 5 of 6)

Table 5-3 (Concluded)

Segment # Station	Region ft, LWD	Year	Cross-sectional Area					Diff. From Template		Number of Observations
			Mean ft^2	Min. ft^2	Max. ft^2	Variance S^2	C.I. ft^2	ft^2	Percent Change	
	-10 to -30	1967	2692	2414	2980	38891[1]	107	+312	+13%	7
		1975	2629	2475	2825	15054[1]	67	+249	+10%	7
		1992	2263[2]	2003	2502	30542[1]	95	-117	-5%	7
	+14 to -30	1967	3963	3704	4235	34660	101	+311	+9%	7
		1975	3806	3615	3975	19973	77	+154	+4%	7
		1992	3298[2]	2975	3625	58560[1]	132	-354	-10%	7

(Sheet 6 of 6)

redistribution of its armor units (outside of the design cross-sectional template) in response to storm wave-, ice-, or foundation-induced events.

Method of comparing cross-section regions - within/across segments. Time-varying aspects for each breakwater segment were examined with respect to cross-sectional changes occurring between 1967 and 1989(92). This type of cross-sectional comparison is referred to as a "within-segment" comparison. The breakwater segments were also compared to one another for each of the three structure surveys (1967, 1975, and 1989). This type of comparison is called an "across-segment" comparison.

The "within-segment" comparison was used to compare the same cross-sectional regions from different surveys within a fixed segment location. This showed how cross sections within a given segment changed with respect to time (same survey location-different survey year). An "across-segment" comparison was used to compare similar cross sections from different breakwater segments, within a fixed temporal interval (different survey location-same survey year). This allowed relative ranking of regions with respect to each other.

Results for the "within-segment" comparison of cross-sectional area **means** are shown in Table 5-4. Each of the three cross-sectional regions within the eight individual breakwater segments is examined in terms of temporal change of the mean cross-sectional area for survey years 1967, 1975, and 1989. Cross-sectional regions (upper/lower/overall) within a fixed segment which are not statistically different from year to year are considered constant, although the mean values may be slightly different. Statistical significance is based upon the Smith-Satterwait procedure for area means (x).

Statistical significance of cross-sectional area change. The (sample) mean values of similar cross-sectional **regions** were compared to each other (via the across-segment and within-segment comparisons) using the Smith-Satterwait (SS) statistical procedure for means comparison. The Smith-Satterwait procedure is a non-parametric method for determining whether or not two sample means are statistically different, for a specified level of confidence. Should two sample means be found to be statistically different, then conclusive inference can be drawn as to whether a particular sample mean is greater than another.

The p-value used in the SS procedure depended upon whether an across-segment comparison or within-segment comparison was being made. P-values indicate the probability of incorrectly drawing inference from a statistical test. The p-value used for within-segment comparisons was 0.25. The p-value used for across-segment comparisons was 0.10. A lower p-value was used for the across-segment comparisons to limit the possibility of making an incorrect inference when comparing cross-sectional area differences between breakwater locations.

Table 5-4
Summary of "Within-Segment" Comparisons

Segment 1 (Station 0+00 - 6+00)		
+14 to -10 ft LWD	**-10 to -30 ft LWD**	**+14 to -30 ft LWD**
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{89} = 1566	X_{89} = 2714	X_{89} = 4281
X_{67} = 1241 = X_{75} = 1226	X_{67} = 2586 = X_{75} = 2579	X_{67} = 3827 = X_{75} = 3805
Segment 2 (Station 7+00 - 16+00)		
+14 to -10 ft LWD	**-10 to -30 ft LWD**	**+14 to -30 ft LWD**
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{89} = 1361	X_{67} = 2480 = X_{75} = 2586 = X_{89} = 2488	X_{89} = 3849
X_{67} = 1206		X_{67} = 3686
X_{75} = 1124		X_{75} = 3591
Segment 3 (Station 17+00 - 22+00)		
+14 to -10 ft LWD	**-10 to -30 ft LWD**	**+14 to -30 ft LWD**
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{89} = 1462	X_{75} = 2508 = X_{89} = 2532	X_{89} = 3994
X_{67} = 1217	X_{67} = 2429	X_{67} = 3646 = X_{75} = 3618
X_{75} = 1110		
Segment 4 (Station 23+00 - 31+00)		
+14 to -10 ft LWD	**-10 to -30 ft LWD**	**+14 to -30 ft LWD**
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{89} = 1464	X_{89} = 2601	X_{89} = 4065
X_{67} = 1237	X_{75} = 2509	X_{67} = 3683 = X_{75} = 3692
X_{75} = 1183	X_{67} = 2446	
Segment 5 (Station 32+00 - 37+00)		
+14 to -10 ft LWD	**-10 to -30 ft LWD**	**+14 to -30 ft LWD**
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{67} = 1273 = X_{75} = 1266 = X_{89} = 1230	X_{75} = 2630	X_{67} = 3812 = X_{75} = 3896
	X_{67} = 2538	X_{89} = 3639
	X_{89} = 2409	

(Continued)

Table 5-4 (Concluded)		
Segment 6 (Segment 38+00 - 46+00		
+14 to -10 ft LWD	-10 to -30 ft LWD	+14 to -30 ft LWD
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{89} = 1400	X_{67} = 2484 = X_{75} = 2502 = X_{89} = 2447	X_{67} = 3749 = X_{75} = 3783 = X_{89} = 3846
X_{75} = 1281 = X_{67} = 1266		
Segment 7 (Segment 47+00 - 50+00		
+14 to -10 ft LWD	-10 to -30 ft LWD	+14 to -30 ft LWD
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{67} = 1294	X_{67} = 2442	X_{67} = 3736
X_{75} = 1184 = X_{92} = 1171	X_{75} = 2371	X_{75} = 3554
	X_{92} = 2132	X_{92} = 3246
Segment 8 (Segment 51+00 - 57+00		
+14 to -10 ft LWD	-10 to -30 ft LWD	+14 to -30 ft LWD
(μ = 1272)	(μ = 2380)	(μ = 3652)
X_{67} = 1271	X_{67} = 2692	X_{67} = 3963
X_{75} = 1177	X_{75} = 2629	X_{75} = 3806
X_{92} = 1045	X_{92} = 2263	X_{92} = 3298
Note: Values which are significantly different are broken out in decreasing order based on P-val of 0.25 or less.		

Applying the SS procedure to breakwater structure surveys of 1967, 1975, and 1989(92) allowed for unbiased ranking of the cross-sectional area means (x) among the eight breakwater segments. After ranking breakwater cross sections in terms of "within-segment" and "across-segment" comparisons, the amount, time interval, and location for which the breakwater experienced structural changes could be determined.

Cross-sectional regions which are statistically different are ranked within the breakwater segment in terms of descending order of cross-sectional area. For example, the cross-sectional area for the upper region (+14 ft to -10 ft (+4.3 m to -3 m) LWD) of Segment 1 did not significantly change between 1967 and 1975, but significantly increased in area from 1975 to 1989. For reference purposes, the expected mean value (μ) of the design template for each cross-sectional region is shown at the top of each segment/region listing. For the upper cross-sectional region, this corresponds to 1,272 sq ft (118 m^2).

Table 5-4 indicates that the breakwater was constructed (1967 survey) very close to the design template area for the upper, lower, and overall cross-sectional regions. In 1975, breakwater Segments 1 and 6 experienced no net

change in cross-sectional area. The lower regions of Segments 3 and 4 gained area at the expense of the upper regions. By 1975, breakwater Segments 2, 7, and 8 had significantly decreased in area from the 1967 condition.

For breakwater Segments 1 through 4 and 6, the present structure condition (1989/92) is either equal to or better (more area) than during any previous survey. This indicates that the breakwater's condition for these segments has been preserved or improved by previous maintenance activities. For breakwater Segments 5, 7, and 8, the present condition is worse than any previous survey year. This implies that past maintenance activities, if any, have not kept pace with the amount of damage sustained by these segments.

Results for the "across-segment" comparisons for mean cross-sectional area are shown in Table 5-5. The eight breakwater segments are ranked relative to each other in terms of the cross-sectional area for each of the three regions (upper/lower/overall). Each breakwater segment is ranked for each of the three survey years; 1967, 1975, and 1989(92). Similar cross-sectional regions originating from different breakwater segment locations are compared (across-segment) and ranked using the SS procedure according to mean cross-sectional area.

Breakwater segments having statistically significant larger cross-sectional regions are ranked "higher" than lower-valued regions. This relative ranking is performed for each of the three survey years (1967/1975/1989), to allow a temporal comparison of individual segments with respect to each other (in terms of mean cross-sectional area). For example, the lower cross-sectional region (-10 to -30 ft LWD (-3 to -9.1-m LWD)) of Segment 5 was ranked in the "midrange" category with respect to the other breakwater segments in 1967. In 1975, the lower region of Segment 5 had risen to the "highest" category. In 1989, the lower region of Segment 5 had fallen back to the "mid-range" category with respect to the other breakwater segments in terms of mean cross- sectional area.

Results from Table 5-5 indicate that for survey year 1967, breakwater Segments 1 and 8 were in the "highest" ranking category with Segments 2 through 4 in the "lowest" category. Note that for 1967, there were no segment regions statistically deficient in area. In 1975, breakwater Segments 1, 5, 6, and 8 were ranked in the "uppermost" category, while 2, 3, and 7 were in the "lower" group. Regions in several segments (2, 3, 4, 7 and 8) were statistically deficient in area. In 1989, Segments 1, 3, and 4 were ranked in the "highest" category, while Segments 5, 7, and 8 were ranked in the "lowest" group. Note that all regions for Segments 7 and 8 are statistically deficient for the 1989(92) survey.

Construction tolerances and cross-sectional variance. Construction tolerances for the completed (as-built) breakwater surface were specified as -1 ft (-0.3 m) (maximum) below the design template neat line. No tolerance was specified with respect to stone placed above the design template neat line. The effect of the -1 ft (-0.3 m) construction tolerance upon the variance for the upper, lower, and overall cross-sections of the breakwater is shown in Table 5-6.

Table 5-5
Summary of "Across-Segment" Comparisons

Survey of 1967	Survey of 1975	Survey of 1989(92)
Upper Region of the Breakwater -> +14 to -10 ft LWD		
(μ = 1272)		
X_7 = 1294 = X_5 = 1273 = X_8 = 1271 = X_6 = 1266 = X_1 = 1241 = X_4 = 1237	X_6 = 1281 = X_5 = 1266 = X_1 = 1226	X_1 = 1566
X_3 = 1217 = X_2 = 1206	X_7 = 1184 = X_4 = 1183 = X_8 = 1177 = X_2 = 1124 = X_3 = 1110	X_4 = 1464 = X_3 = 1462
		X_6 = 1400 = X_2 = 1361
		X_5 = 1230 = X_7 = 1171
		X_8 = 1045
Lower Region of the Breakwater -> -10 to -30 ft LWD		
(μ = 2380)		
Survey of 1967	**Survey of 1975**	**Survey of 1989(92)**
X_8 = 2692	X_5 = 2630 = X_8 = 2629	X_1 = 2714
X_1 = 2586		X_4 = 2601
X_5 = 2538	X_1 = 2579 = X_4 = 2509 = X_3 = 2508 = X_6 = 2502	X_3 = 2532 = X_2 = 2488
X_6 = 2484 = X_2 = 2480 = X_4 = 2446 = X_7 = 2442 = X_3 = 2429	X_2 = 2467	X_6 = 2447 = X_5 = 2409
	X_7 = 2371	X_8 = 2263
		X_7 = 2132
Overall Region of the Breakwater -> +14 to -30 ft LWD		
(μ = 3652)		
Survey of 1967	**Survey of 1975**	**Survey of 1989(92)**
X_8 = 3963	X_5 = 3896 = X_8 = 3806 = X_1 = 3805 = X_6 = 3783	X_1 = 4281
X_1 = 3827	X_4 = 3692	X_4 = 4065 = X_3 = 3994
X_5 = 3812 = X_6 = 3749 = X_7 = 3736	X_3 = 3618 = X_2 = 3591 = X_7 = 3554	X_2 = 3849 = X_6 = 3846
X_2 = 3686 = X_3 = 3646 = X_4 = 3683		X_5 = 3639
		X_7 = 3246 = X_8 = 3298

Note: Values which are significantly different are broken out in decreasing order based on a P-val of 0.10 or less.

Table 5-6
Acceptable Variability of the Specified Construction Template for Breakwater Cross-Sectional Regions

Cross Section	Tolerance of Constructed Cross-Sectional Area Below Design Template (- sq ft)	Percent Difference From Template (-)	Expected Variance[1] Based on Tolerance
Upper	86	7	3,767
Lower	72	3	2,640
Overall	158	4	12,716

[1] Based on t-distribution using sample size of 7.

Construction tolerances conceptually represent design template confidence intervals for each cross-sectional region. If an as-built (1967) cross section had an area less than the "design cross section minus the design confidence interval," then the as-built cross section is considered to be statistically deficient with respect to the design template. This type of argument can also be applied to any of the other breakwater surveys, with respect to the design template tolerances shown in Table 5-6.

The analogy of the design template confidence interval can be applied only in cases where the surveyed breakwater cross-sectional area falls below the design template (μ), since only negative construction tolerances were specified for the design template. For example, if the average cross-sectional area of the upper region of a particular breakwater segment is less than 1186 sq ft (110 sq m) (1,272-86 sq ft (118-8 sq m)), then the upper region of that segment is considered statistically deficient, in terms of area, for the survey of interest. This type of comparison was performed for each of the three structure surveys (1967, 1975, and 1989(92)). Results are shown in the fourth column of Table 5-3. The numeral 2 beside values in column 4, Table 5-3, indicates that the respective cross-sectional region is statistically less than the design template.

The values shown in column 1 of Table 5-6 were used as detection thresholds in the section of this report entitled "Breakwater Design Performance." The actual confidence interval (CI, 80 percent) for cross-sectional area x was determined for each breakwater cross section and is shown in column 8, Table 5-3. Comparison of the CIs between similar cross-sectional regions originating from different breakwater segments illustrates the degree of variability (uncertainty) associated with the mean value area x for each cross- sectional region. Significant overlapping of cross-sectional means ($x \pm$ CI), between similar cross-sectional regions from different surveys indicates that the means are equivalent for different survey years. This is the basic principal behind the Smith-Satterwait procedure for comparing cross-sectional means x.

Variance of the cross-sectional area. The variance (s^2) associated with the specified construction tolerance has been determined for the upper, lower, and overall cross-sectional regions of the breakwater profile (column 4,

Table 5-6). These "expected" variances are based on a sample size of 7, which is characteristic of the breakwater segments. Design template variance was calculated in terms of the 98-percent confidence intervals for each cross-sectional region.

In the case of survey data, large variances for cross-sectional area are related to the difference between the minimum and maximum cross-sectional area for a given region within a breakwater segment of interest. The greater the difference between minimum and maximum cross-sectional area within a given region, the larger the cross-sectional variance (variability) for that region. The variance (or confidence interval) of a particular breakwater region was used to gauge the degree of irregularity present within the region of interest.

Relative differences in the cross-sectional variance (from year to year within a given region) of greater than four times are considered to constitute a <u>significant</u> change in cross-sectional variance and hence the configuration of the cross-sectional region in question. This was determined from an F-test of variances with a sample size of 7 and a rejection threshold of 0.9. The F-test is a standard statistical test of significance for two variances.

For example, if an upper region of a breakwater segment had a variance of 3,000 (for cross-sectional area) in 1967 and of 13,000 in 1975, it could then be concluded that this region of the breakwater had experienced a significant change in configuration with respect to the design template (variance of 3,767 or less, Table 5-6).

This type of comparison was performed for each of the three structure surveys (1967, 1975, and 1989(92)). Results are shown in column 7 of Table 5-3. The numeral 1 beside values in column 7 indicates that the respective cross-sectional region is statistically more irregular than: (a) the previous survey year, for the same segment region, or (b) the design template. Note that for 1989, segments 2, 6, and 8 display a significant increase in cross-sectional variance from the previous survey years (1967 or 1975).

Attention was drawn to the segments of the breakwater which exhibited high cross-sectional variance. The difference in the variance magnitude between upper and lower cross-sectional regions, at highly variable segments, supports the methodology of using the -10 ft (-3 m) LWD elevation as the cut-off point for partitioning the overall breakwater cross section into upper and lower regions. The degree of variability of an overall cross section can be attributed to either one or both of the upper and lower cross sections.

Correlation of cross-sectional area mean and variance. The structural response of a rubble-mound breakwater subjected to environmental events which meet or exceed the structure's design constraints is manifested in a transition from a uniform (constructed) cross section to one which is highly variable with respect to a prescribed design template. The cross section of a hypothetical rubble-mound breakwater which was subjected to exceedingly large environmental forces over time would show a trend of increasing

cross-sectional variability with time. Due to the unintentional undersizing of armor stone (through the under-estimation of design wave height) on the Burns Harbor breakwater, the overall cross section was designed/constructed in a manner which promoted the redistribution of armor stone to a more stable (but more spatially variable) configuration with passage of time.

While a sudden increase/decrease in breakwater variability alone may not warrant attention, an increase in variance coupled with a significant change in cross-sectional area may indicate that an undesired process (damage) or desired effect (repair) is under way. Comparison of variances (s^2) and mean cross- sectional areas (x) for all eight breakwater segments resulted in the formulation of the following four general hypotheses to account for variance and cross-sectional area trends.

a. *Trend 1*. Small variance change accompanied by a significant decrease in cross-sectional area is speculated to be the result of en masse breakwater settlement.

b. *Trend 2*. Large variance change accompanied by a significant decrease in cross-sectional area is assumed to be the result of numerous slope instabilities induced either by localized settlement or wave/ice damage.

c. *Trend 3*. Small variance change accompanied by a significant increase in cross-sectional area is speculated to be the result of successful widespread and uniform breakwater repair.

d. *Trend 4*. Large variance change accompanied by a significant increase in cross-sectional area is assumed to be indicative of localized and nonuniform breakwater repair.

Of the four possible breakwater responses, Trend 2 is assumed to represent the worst-case scenario for breakwater damage. A rubble-mound breakwater, constructed with large blocky (regular-shaped) armor units, subjected to highly random damage (stone redistribution) is susceptible to further deterioration due to the unraveling effect associated with such armor units. Conversely, the most desirable breakwater response would be Trend 3.

According to differences between the 1967, 1975, and 1989(92) surveys, breakwater Segments 5 and 8 have experienced a Trend 2 response. Segments 1, 3, and 4 show Trend 3 response. Only Segment 7 shows a Trend 1 response, with Segment 2 showing a Trend 4 response. Segment 6 shows no significant change with respect to mean cross-sectional area or variance between 1975 and 1989, despite 17,600 tons (15,800 mt) of stone being placed on that portion of the breakwater for the same time period.

Maintenance effectiveness and stone "loss"

Cross-sectional change of the breakwater (presented in terms of stone quantities) was compared with the actual amount (tons) of maintenance stone placed on the breakwater, with the results of which are shown in Table 5-7.

Table 5-7
Change of the Breakwater Cross Section in Terms of Tons of Stone Lost or Gained per Breakwater Segment for 1975-1989(92)[1]

Segment No.	Length Lin. ft	Estimated Amount of Stone Gained/Lost[2]			Actual Amount Placed Net Total tons	Difference/ft est. - act. tons/ft	Rank
		Upper Region tons	Lower Region tons	Net Total tons			
1	600	+10,526	+4,180	+14,737	34,524	-33.0	1
2	900	+9,124	N/A (0)[3]	+9,932	24,556	-16.2	7
3	500	+7,528	N/A (0)	+8,042	15,943	-15.8	8
4	800	+9,616	+3,148	+12,764	26,301	-16.9	6
5	500	N/A (0)	-4,727	-5,497	7,313	-25.6	2
6	800	+4,072	-1,882	+2,156	17,585	-19.3	5
7	300	N/A (0)	-3,067	-3,952	3,510	-24.9	3
8	600	-3,388	-9,393	-13,038	0	-21.7	4

[1] Differences are in terms of lineal foot of breakwater; lower numbered rank designates a more severe stone loss/attrition rate.

[2] Determined from change in average area of cross-sectional region within the segment of interest based on: average area difference (1989(92) - 1975) × segment length × 0.59 × 145 pcf/2,000 pounds per ton. For Segment 1, 0.65 was used for percent solids and 150 pcf was used for specific weight

[3] N/A = Statistically no change in segment cross-sectional area for a particular region.

An "N/A" entry is made for breakwater segments which did not experience statistically significant cross-sectional change between 1975 and 1989(92).

Note that entries shown in the "DIFFERENCE" column of Table 5-7 indicate relative change of stone quantity within each breakwater segment between 1975 and 1989: A negative value indicates effective stone loss; and a positive number indicates effective stone gain. These values include the maintenance stone placed on the breakwater. Therefore, a positive value in the "DIFFERENCE" column is unrealistic since it would indicate a gain in stone quantity over and above that which was placed on the breakwater during maintenance activities.

For example, the overall cross-sectional region of Segment 1 experienced an estimated gain of 14,737 tons (13,370 mt) of stone between 1975 and 1989. However, 34,524 tons (31,340 mt) of stone were placed on Segment 1 during this same period. Based on the area difference between the 1989 and 1975 surveys and accounting for placed maintenance stone, Segment 1 had "lost" 19,787 tons (17,950 mt) of armor stone (or 33 tons/lin ft (98.4 mt/m). At a typical armor stone size of 13 tons (12,000 mt), this represents a loss of about 2 stones per foot (0.3 m). The term "lost" infers that the stone has been displaced below -30 ft (-9.1 m) LWD or has fallen completely off of the breakwater cross section. In either case, the "lost" stone serves no functional purpose.

Individual breakwater segments are ranked according to stone loss per lineal foot in the last column of Table 5-7. Lower numerical entries indicate a more severe (stone loss) condition. Breakwater Segments 1 and 5 represent the most severe locations along the breakwater for stone loss/attrition rates. Segments 6, 7 and 8 have experienced loss rates almost as high as Segments 1 and 5. The high stone loss for breakwater Segments 7 and 8 was not expected, given that this portion of the breakwater is situated at a less severe wave exposure orientation than Segments 1 through 6. The stone loss from Segments 7 and 8 may be attributable to foundation settlement, ice-induced forces, or a damage mode not addressed here. Segments 2, 3, and 4 are relatively similar in the amount of stone lost since 1967, representing roughly one half the stone loss per lineal foot of Segments 1 and 5.

Conclusions

Although these conclusions are based on the cross-sectional analysis of Burns Harbor breakwater, they may also be applied to forensic design of other riprap (rubble-mound) maritime structures.

Conducting surveys for large armor rubble-mound breakwaters. Unfortunately, some survey data (cross-section spot elevations) were omitted during the 1967 and 1989 surveys, making systematic analysis from survey to survey problematic. Riprap breakwater surveys should be conducted in such a manner as to afford complete cross-section coverage. Spot elevations should be taken and recorded along pre-defined intervals and at surface break points. Strict field quality control should be implemented to avoid "holes" or "data

omitted" areas along each cross section. This may be challenging near the waterline interface during marginal weather conditions. Objective surveying is required if objective analysis is to be performed using such data.

Fathometer-induced bias of riprap soundings. Recent experience with the sounding of steep rubble-mound slopes using a fathometer has shown acoustic soundings to be susceptible to spatial distortion. When sounding steep rubble-mound slopes with a fathometer, acoustic signals will rebound from the stones closest to the signal source (i.e. stones positioned at an angle higher up the slope). Thus, the sonic equipment will register, at points of survey, depths which are shallower than true vertical depths.

Consequently, profiles obtained from such biased data will show a swelled configuration of the rubble-mound structure, with the base of the breakwater wider than actual, and slopes flatter than actual. This acoustic distortion is not always inherent in rubble-mound soundings and can be minimized or controlled if proper equipment and tuning methods are utilized during a fathometer-based sounding. Acoustic distortion may be exaggerated when large armor units are present. Distortion can be reduced using the following techniques:

a. Employment of a narrow beam (8 deg or less) transducer featuring depressed sidelobes to minimize distortion caused by steep slopes.

b. Use of reduced/tuned "gain" settings to inhibit the corruption effect of highly irregular surfaces up/down the slope of the desired survey point.

It is suggested that future surveys of the Burns Harbor breakwater utilize mechanical sounding methods (lead line and sounding basket or sounding pole). Without consistently incorporating stringent quality control to verify acoustic soundings, fathometer-based structure surveys of the breakwater are susceptible to an inherent degree of uncertainty. This finding may be applicable to other breakwaters in which the armor layer is composed of larger blocky stone (nominal dimension of 5.5 ft (1.7 m)) (McGehee 1987).

Adherence to pre-defined survey control. The rubble-mound breakwater at Burns Harbor has been surveyed throughout its life-cycle according to three baseline/center line configurations. Horizontal controls used for the layout of each baseline or center line were not cross-referenced, through field data, to one another. Since each structure survey was performed using a different horizontal control layout, consistent (horizontal) comparison between two different surveys was not possible. This precluded assessing breakwater damage from the standpoint of harborside versus lakeside trends. Harborside damage is considered an important stability factor for this case due to significant and chronic wave-overtopping of the breakwater.

A vertical method of cross-sectional analysis was used, in place of a horizontal method, to assess the time-varying characteristics of Burns Harbor breakwater. This permitted using unbiased statistics of mean cross-sectional

area and variance to comparatively assess the breakwater for survey years 1967, 1975, and 1989(92). The vertical basis of cross-sectional analysis can be systematically applied to any structure which is designed, constructed, or maintained according to a specific cross-sectional template.

Had horizontally consistent surveys been conducted from the beginning of the breakwater's life cycle, changes in the structure's cross section could have been monitored with respect to the crest center line. It is recommended that time-invariant and consistent horizontal control be implemented for future surveys of Burns Harbor breakwater.

Assessment of cross-sectional area change. All segments/regions of the breakwater were constructed either at or above required specifications, according to the within- and across-segment comparisons for the 1967 survey. By 1975, however, the breakwater had experienced notable damage (diminished cross-sectional area) in some locations.

Segments 2, 3, and 7 were considered deficient in cross-sectional area according to the across-segment comparison of the 1975 survey. In 1989(92), breakwater Segments 1 through 6 were at or above the minimum cross-sectional area requirements. This is due to placement of 145,000 tons (132,000 mt) of maintenance stone during 1975-1989. However, Segments 7 and 8 were significantly deficient in cross-sectional area as compared to the design template.

Burns Harbor breakwater exhibited a highly irregular configuration for Segments 2, 6, and 8 according to the 1989(92) survey as illustrated by the high variance documented in Table 5-3. This is indicative of cumulative effects of random breakwater damage and incremental repair activities from 1975 to 1989.

Breakwater repair efficiency. An efficient repair process is considered to be one which results in a long-term solution (fix) to a given damage trend. Breakwater Segments 1, 5, 7, and 8 have experienced extremely poor maintenance efficiency between 1975 and 1989(92). In other words, the maintenance stone placed on these segments can not be accounted for. Segments 5 and 7 "lost" more stone than was placed on these breakwater locations. Only one half the stone tonnage placed on Segment 1 is accountable on the structure surveys. Segment 3 had the best repair efficiency, with a stone loss rate one half that of Segments 1 and 5.

The breakwater's poor maintenance efficiency may be due to a damage mechanism caused by waves/ice, foundation settlement, or both. In either case, maintenance/repair activities have not adequately addressed the breakwater's continual damage trend. Most of the damage sustained by the rubble-mound breakwater at Burns Harbor occurred from 1975 to 1989(92), which coincides with the fact that all maintenance for the breakwater occurred between 1975 and 1989.

Breakwater Design Performance

Breakwater design performance was separated into two issues. The first issue dealt with identifying the location and magnitude of cross-sectional area change occurring over the life cycle of the structure. The second issue involved determining if those changes are explainable using present deterministic theoretical practices. Cross-sectional analysis using breakwater surveys provides a method of measuring actual changes along the breakwater within the tolerance of survey error. Analyses were conducted to deterministically quantify the changes in the breakwater predicted by theory. For the Burns Harbor breakwater, two modes of damage were examined: settlement and wave damage.

Wave climate at Burns Harbor breakwater

Wave history. To approximate the storm wave heights at Burns Harbor breakwater, Wave Information Study (WIS) deepwater wave heights were transformed to the shallower breakwater site using the TMA wave transformation theory described in Hughes 1984. This wave transformation theory requires parameters of energy-based significant wave height, peak period and dominant direction, and wind speed and direction. Wind data used in the TMA transformation were from the WIS data set.

To isolate pertinent storm data, a search was conducted of the WIS data for specific storm criteria at the Burns Harbor location. A storm was defined as the occurrence of wind speeds of 20 mph (32 km/hr) or greater for a minimum of 9 hr from either 315 to 360 deg or 0 to 45 deg, where 0 deg is north, 90 is east, etc. The information was provided for Station 62 from the WIS data grid (Figure 5-5). For the 32 years of record, wind and wave information were compiled for storm events that met these criteria (Hubertz, Driver, and Reinhard 1991).

The WIS plotted the total number of storms (as defined above) occurring each year at this station for the period of record (1956-1987) and the total number of storms occurring in each month for all years. Illustrated in Figure 5-6, those plots indicate the "stormy" years and months at this location. As can be seen from the storms per year plot, storms falling under the above criteria are relatively common with an average of 12 storms per year occurring. January, February, and March are the most severe storm months.

From the storm information furnished by the WIS data set, the maximum 32 storm events were chosen in terms of deepwater significant wave height. Comparison of the WIS data set with the wave gage data indicated a consistent bias in the WIS wave periods. This bias was determined to be a 2-sec underestimation of the peak wave periods.

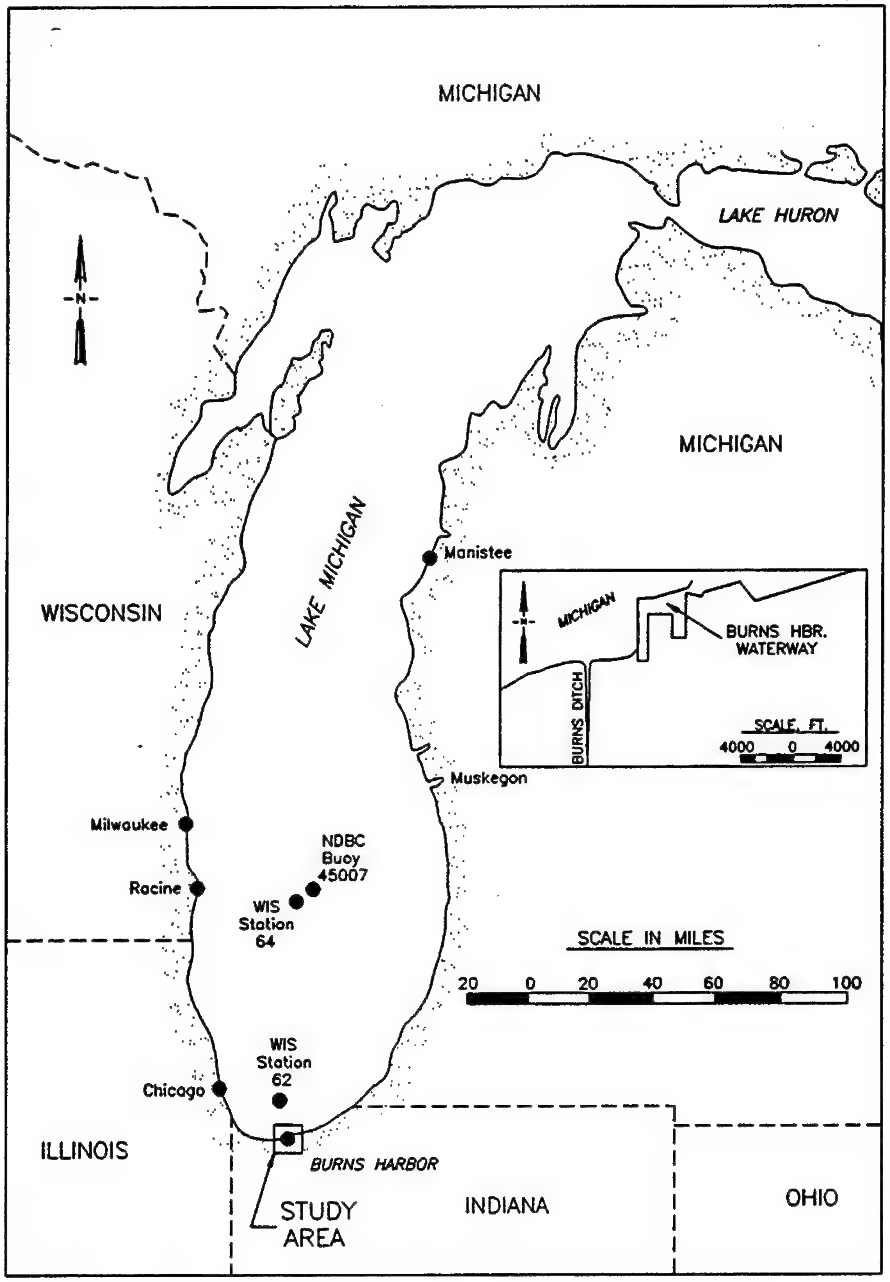

Figure 5-5. Location of WIS Station 62

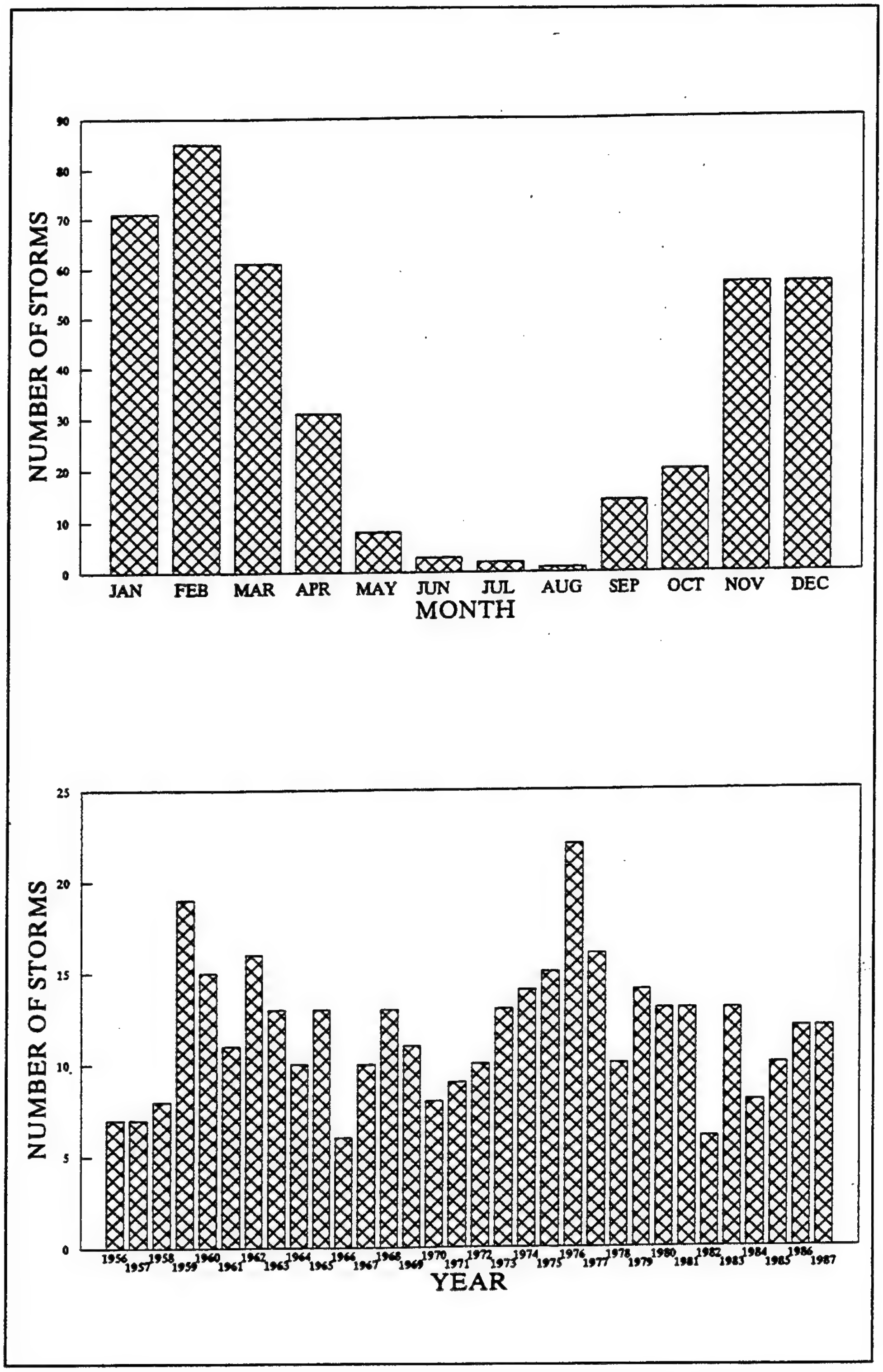

Figure 5-6. Storm months and years for Station 62, Lake Michigan

Table 5-8 lists parameters of the WIS maximum storm events. The wave period has been adjusted upward by 2 sec to correct for bias. The wind speed listed represents the average of the three highest 3-hr intervals during the storm event. Table 5-9 lists return interval versus deepwater wave height at Station 62 as documented from Hubertz, Driver, and Reinhard 1991. Figure 5-7, which is a storm history, illustrates how those storm events are distributed over the 32-year period of record.

Wave Information Study (WIS) deepwater data (1956-1987) were transformed to the structure site using the TMA wave transformation theory. The first step in this wave transformation analysis was a comparison of the TMA-transformed wave heights to the measured wave heights at the gage. Due to the extremal range of wave heights and the relatively short (1985-1987) period of record of wave gage data, only seven events were available for comparison. Table 5-10 lists the parameters of those seven events as given by the wave gage data.

The gage wave height measurements listed were Site 1 measurements (with the exception of the December 1987 event which was Site 4. All wave height values have been corrected for reflection from the breakwater. Table 5-11 lists the WIS parameters for the seven storm events. Wave periods have been adjusted for bias.

Hughes (1984) was used as a reference in the transformation of the WIS data to the site of the wave gage. The water depth as measured by the wave gage for each event was used in the TMA analysis. The wind speed used was the average of the three highest 3-hr intervals for the storm event.

The wave height factor of 1.1 was not used due to the close approximation already provided by the TMA-estimated wave. Table 5-12 compares the TMA-transformed WIS wave to the wave which was measured by the wave gage. Wave heights compared are energy-based H_{mo}. Figure 5-8 plots the two wave height estimates versus the gage wave period.

The TMA transformation of the WIS deepwater data compares well with the measured wave heights for the seven storm events analyzed with an average difference of 0.5 ft (0.2 m). Due to the close approximation of the TMA-predicted wave heights, the TMA theory was used to transform the other significant storm events into the structure. A partial duration frequency analysis was performed on the transformed wave heights at the structure. Figure 5-9 was used to convert the transformed WIS data from H_{mo} to H_S for the rubblemound stability analysis. Table 5-13 lists TMA-transformed storm events used in the frequency analysis. A frequency distribution for incident wave height at the structure was developed using the Weibull probability distribution as shown in Figure 5-10.

Table 5-8
Significant Storm Events-WIS Data Set (Hubertz, Driver, and Reinhard 1991)

Date	Deepwater Significant Wave ft	Peak Wave Period sec	Wind Speed mph	Rank
11 Dec 57	12.5	11.1	32.0	23
22 Jan 59	13.4	11.1	32.0	15
09 Mar 61	13.1	11.1	29.1	17
30 Jan 62	11.5	10.3	32.0	31
13 Jan 64	12.1	9.7	32.9	24
25 Feb 65	19.0	12.0	40.3	1
25 Dec 65	14.1	11.1	36.5	10
29 Nov 66	15.1	12.0	35.8	7
27 Jan 67	13.8	11.1	35.1	14
15 Dec 68	12.1	10.3	32.9	25
30 Nov 71	11.5	10.3	31.3	32
14 Nov 72	12.5	10.3	30.6	22
29 Jan 73	14.1	12.0	34.2	11
15 Feb 73	12.1	11.1	29.1	26
18 Mar 73	12.8	12.0	26.2	20
22 Feb 74	14.4	11.1	33.6	9
13 Nov 75	13.1	10.3	31.3	18
01 Feb 76	13.8	11.1	40.9	13
22 Feb 76	15.4	11.1	35.1	6
20 Dec 76	11.8	10.3	34.2	30
14 Jan 79	13.4	11.1	31.3	16
26 Feb 79	13.8	10.3	32.9	12
25 Dec 79	15.4	12.0	37.4	5
26 Feb 80	12.1	10.3	33.6	29
02 Dec 80	12.8	11.1	29.8	19
24 Dec 80	12.5	11.1	32.0	21
20 Nov 81	12.1	11.1	27.5	28
11 Nov 83	15.1	12.0	34.2	8
16 Nov 83	12.1	12.0	26.8	27
28 Feb 84	16.4	12.0	35.8	4
12 Feb 85	17.1	12.0	35.1	3
08 Feb 87	17.7	12.0	41.8	2

Table 5-9 Deepwater Wave Height Versus Return Interval (Hubertz, Driver, and Reinhard 1991)	
Return Interval **years**	**Wave Height, H_s** **ft**
5-year	16.1
10-year	17.1
20-year	18.0
50-year	19.4

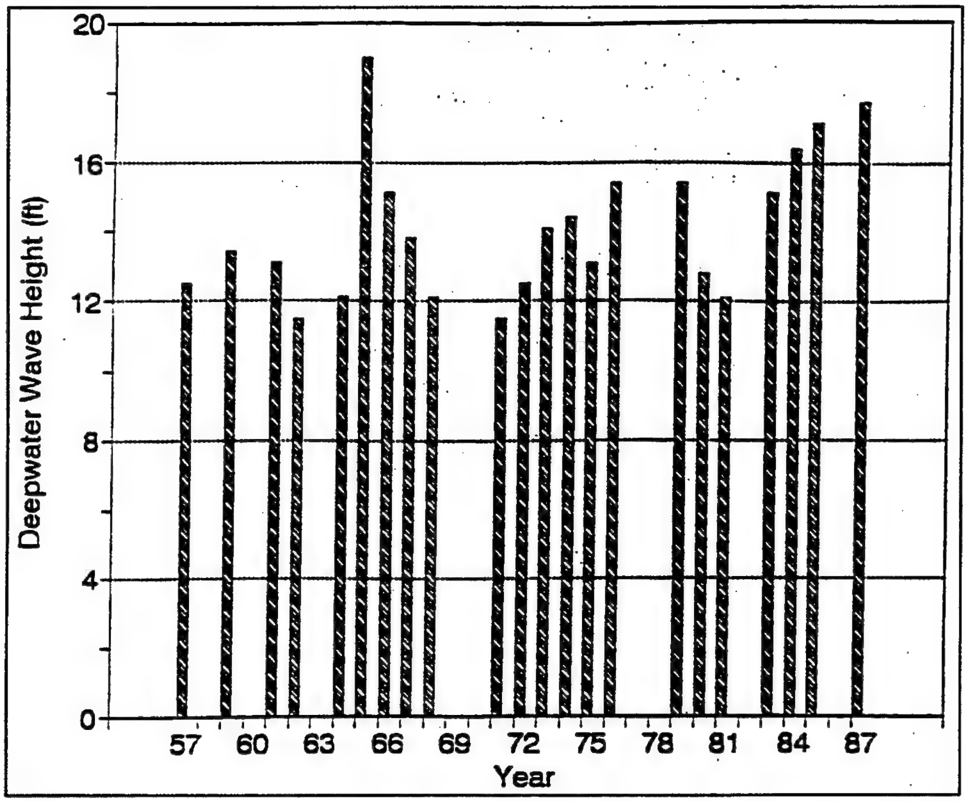

Figure 5-7. Storm history, 1957 to 1987

Table 5-14 gives the significant wave height ($H_{1/3}$) at the structure versus return interval. Figure 5-10 plots $H_{1/3}$, $H_{1/5}$, and $H_{1/10}$ versus return interval for the Burns Harbor site. In addition, the original $H_{1/3}$ wave height distribution which was used in the design is plotted.

Table 5-10
Wave Gage Parameters for Transformed Events

Date	Wave Height (H_{mo}) ft	Wave Period sec	Water Depth ft
24 Dec 85	8.2	9.9	46.2
13 Jan 86	8.9	9.9	46.9
23 Jan 87	7.5	9.2	46.6
08 Feb 87	14.1	11.6	47.9
09 Mar 87	10.5	10.7	47.6
05 Apr 87	7.5	7.1	46.6
04 dec 87	8.5	7.5	49.5

Table 5-11
WIS Parameters for Transformed Events

Date	Deepwater Wave Height ft	Wave Period sec	Wind Speed mph
24 Dec 85	7.9	8.7	24.6
13 Jan 86	8.2	8.3	28.4
23 Jan 87	8.5	8.7	26.9
08 Feb 87	17.7	12.0	41.8
09 Mar 87	10.2	9.1	31.3
05 Apr 87	8.9	7.9	23.1
04 Dec 87	5.9	8.3	21.7

Table 5-12
TMA-Transformed WIS Wave Height Versus Gage Wave Height

	Wave Gage		TMA-Transformed WIS	
Date	Wave Height ft	Wave Period sec	Wave Height ft	Wave Period sec
24 Dec 85	8.2	9.9	8.8	8.7
13 Jan 86	8.9	9.9	9.2	8.3
23 Jan 87	7.5	9.2	9.2	8.7
08 Feb 87	14.1	11.6	14.8	12.0
09 Mar 87	10.5	10.7	10.4	9.1
05 Apr 87	7.5	7.1	8.0	7.9
04 Dec 87	8.5	7.5	8.2	8.3

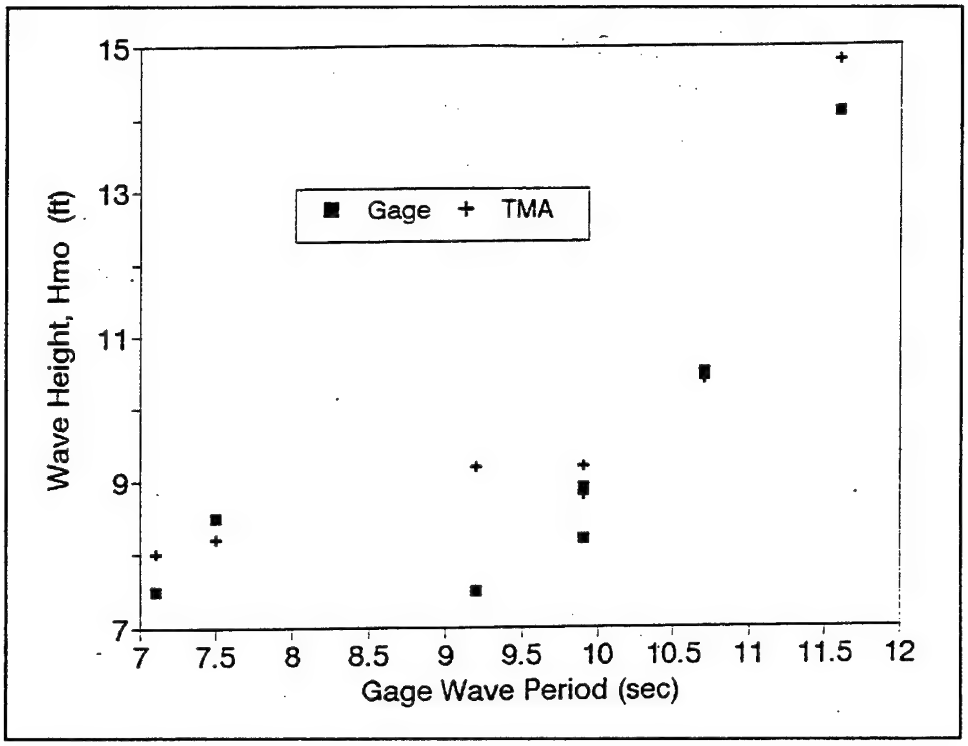

Figure 5-8. TMA-transformed waves versus gage waves

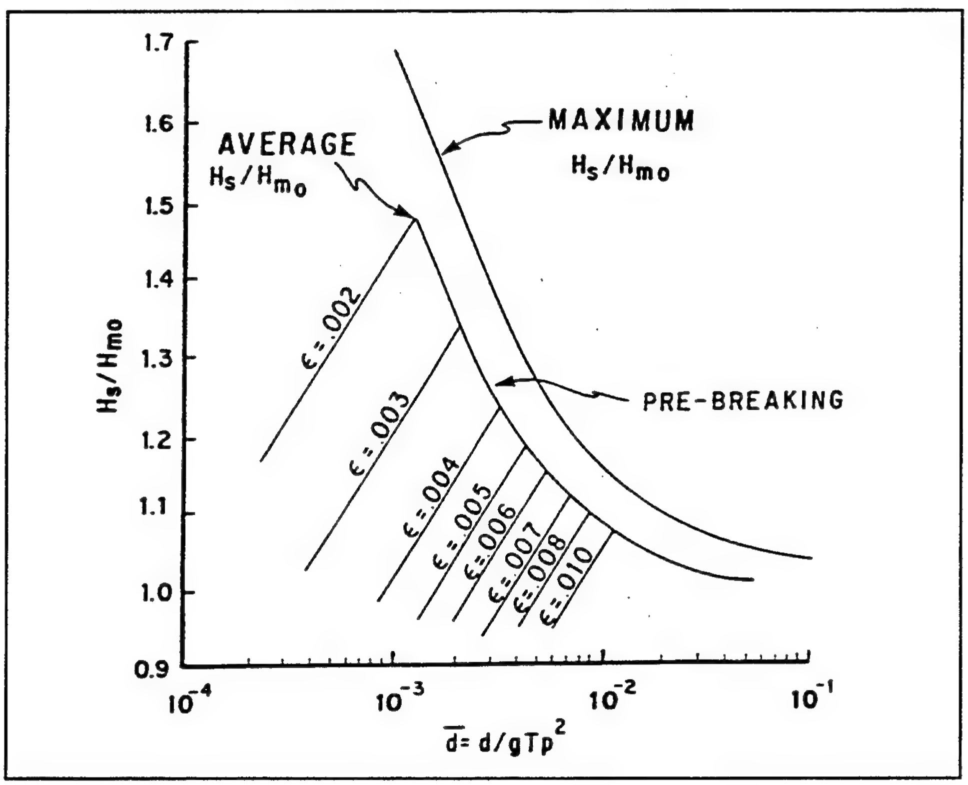

Figure 5-9. H_{mo} to H_S conversion relationship

Table 5-13
TMA-Transformed Wave Heights

Date	Wave Height (H_{mo}) ft	Wave Height (H_s) ft	Period sec
11 Dec 57	11.8	13.5	11.1
22 Jan 59	11.8	13.5	11.1
09 Mar 61	11.2	12.9	11.1
30 Jan 62	11.1	12.7	10.3
13 Jan 64	10.8	12.2	9.7
25 Feb 65	14.0	16.4	12.0
25 Dec 65	12.6	14.4	11.1
29 Nov 66	13.2	15.4	12.0
27 Jan 67	12.3	14.2	11.1
15 Dec 68	11.3	12.9	10.3
30 Nov 71	11.0	12.6	10.3
14 Nov 72	10.9	12.4	10.3
29 Jan 73	12.9	15.1	12.0
15 Feb 73	11.2	12.9	11.1
18 May 73	11.3	13.2	12.0
22 Feb 74	12.1	13.9	11.1
13 Nov 75	11.0	12.6	10.3
01 Feb 76	13.3	15.3	11.1
22 Feb 76	12.3	14.2	11.1
20 Dec 76	11.5	13.1	10.3
14 Jan 79	11.7	13.4	11.1
26 Feb 79	11.3	12.9	10.3
25 Dec 79	13.5	15.8	12.0
26 Feb 80	11.4	13.0	10.3
02 Dec 80	11.4	13.1	11.1
24 Dec 80	11.8	13.5	11.1
20 Nov 81	10.9	12.6	11.1
11 Nov 83	12.9	15.1	12.0
16 Nov 83	11.5	13.4	12.0
28 Feb 84	13.2	15.4	12.0
12 Feb 85	13.1	15.3	12.0
08 Feb 87	14.2	16.7	12.0

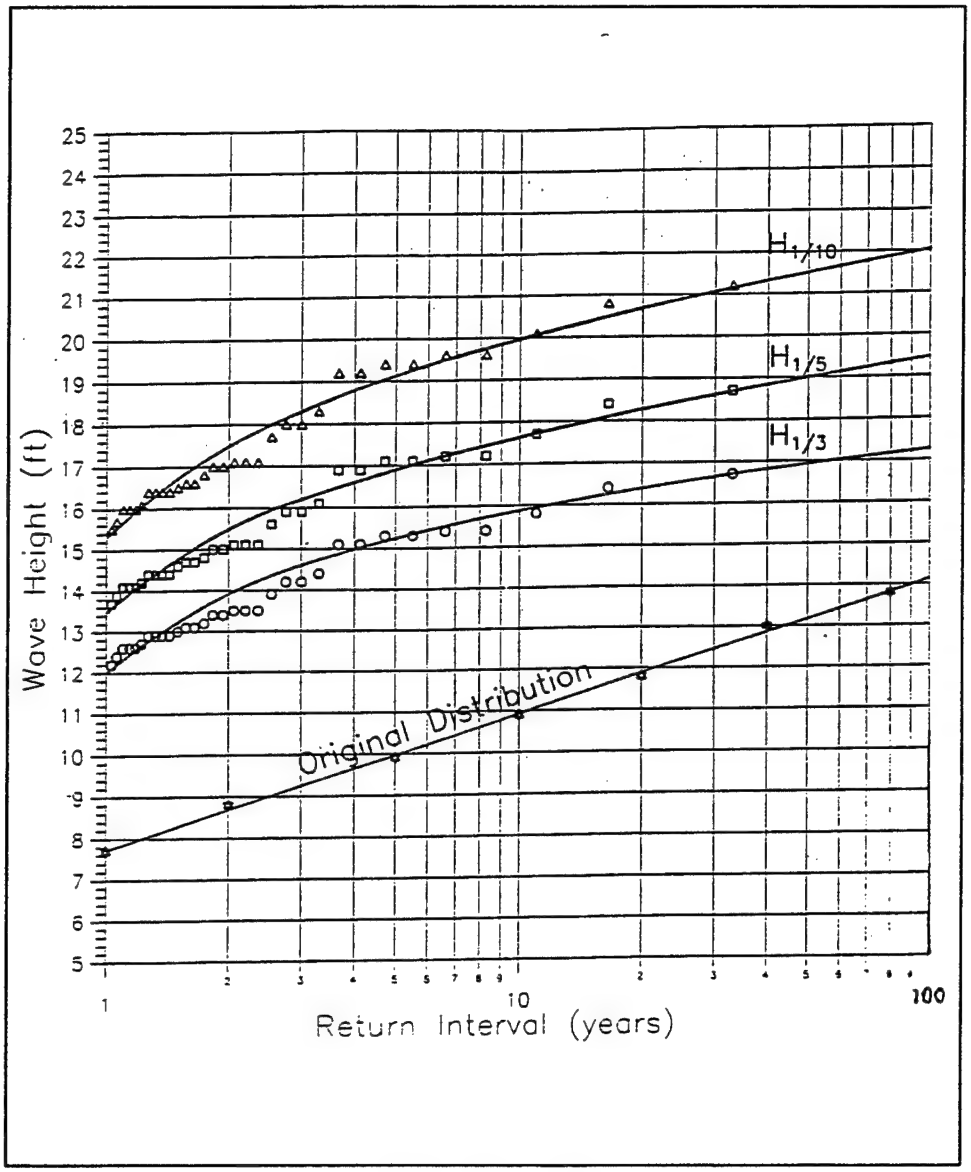

Figure 5-10. Return interval versus wave height

Table 5-14
Significant Wave Heights at the Structure

Return Interval years	Wave Height, H_s ft
5-year	15.3
10-year	15.8
20-year	16.3
50-year	16.9

This wave climate analysis indicates that both the 13-ft (4-m) design wave used in the original Burns Harbor design as well as the 15-ft (4.6-m) design wave established in the model study were smaller than even the 5-year significant wave height at the structure. The 11-ft (3.4-m) wave height used for the overtopping design has a frequency of less than 1 year.

Since a 50-year return interval significant wave at the site was determined to be 16.9 ft, the 5- to 20-ft (1.5- to 6-m) wave height range used in the 2-D model study was a reasonable design wave climate. Over the time period extending from project completion in 1967 to 1987, the breakwater has been exposed to 15 storm events exceeding the 13-ft (4-m) significant wave height and seven storm events exceeding the 15-ft (4.6-m) wave height.

Design-predicted damages

During the design of the breakwater, damages were predicted to be negligible over the life of the structure. Settlement was not considered to be a potential damage mode due to the proposed treatment of the foundation. Damages due to waves were estimated based on rubble-mound stability tests conducted by R. Y. Hudson at the U.S. Army Engineer Waterways Experiment Station. Predicted damages were calculated as a function of a given ratio of wave height above the design wave.

Results of the physical model study conducted in 1967 for the Burns Harbor breakwater indicated that the structure was stable for 15-ft (4.6-m) waves (Jackson 1967). Another way of characterizing the stability results of the model study is to say that the stability coefficient K_D was estimated to be 5.4 for a 15-ft (4.6-m) design wave. The original damage analysis performed by Sverdrup and Parcel indicated that for the selected design wave height of 4.6 m (15 ft), there would be no maintenance required for the breakwater.

Quantification of settlement damages

The final breakwater design in 1965 recommended removal of soft clay up to 20 ft (6 m) thick from the lakebed and backfilling with sand to a specified depth. It was expected that this measure would prevent any substantial settlement and subsequent damages to the breakwater.

The Chicago District's Geotechnical and Coastal Branch reinvestigated the foundation conditions and potential settlement of the structure. Results of this deterministic investigation can be found in Appendix 5C. The foundation investigation was conducted using the same breakwater partitioning that was used for the cross-sectional analysis portion of this report to allow for a comparison of results. Using the predicted settlement areas and their applicable lengths along the breakwater, potential volume and tonnage of stone were calculated.

Table 5-15
Predicted Cross-Sectional Area Loss Due to Settlement

Breakwater Segment	Segment Length	Potential Settlement		Area Loss, ft^2		
		Center Line ft	Toe ft	Minimum	Mean	Maximum
1	650	1.08	0.32	87	173	260
2	1000	1.90	0.58	152	304	456
3	600	1.66	0.38	129	257	386
4	900	1.03	0.33	83	166	249
5	600	0.80	0.23	64	127	191
6	900	1.04	0.33	84	167	251
7	400	1.21	0.42	99	197	296
8	700	1.43	0.43	114	228	342

Calculations were also performed to determine the amount of additional settlement that may have occurred from placement of the maintenance stone since the breakwater's construction. EM 1110-1-1904, "Settlement Analysis," states that settlement predictions are accurate to within 50 percent of actual settlements. Settlement calculations were estimated to be accurate to ± 50 percent of the estimate.

Table 5-15 lists, by segment, the range in the predicted loss of cross-sectional area due to settlement. Quantities presented in Table 5-15 may differ slightly from values given in Appendix 5C since the values in the table represent weighted averages by segment.

Figure 5-11 illustrates the expected distribution of cross-section adjustment due to settlement. This typical section distributes 64 percent of the area loss in the upper region and 36 percent in the lower region. As can be seen from this figure, potential settlement could result in minor elevation adjustments along the full cross section. This type of cross-section adjustment is considered normal for a structure this size. A 1-ft (0.3-m) magnitude settlement represents approximately 1/40 of the structure height and 1/6 of a typical armor stone dimension.

An estimate of the time rate of consolidation was also performed and is illustrated in Figure 5-12. The trend shown in Figure 5-12 suggests that, to date, the breakwater has undergone an estimated 80 percent of its total anticipated settlement. Since experience suggests that the consolidation theory used in this analysis represents an upper limit, it was assumed that essentially 100 percent of consolidation had occurred by 1993[1].

[1] Personal communication, 1993, John Fornek, Geotechnical Engineer, U.S. Army Engineer District, Chicago, Chicago, IL.

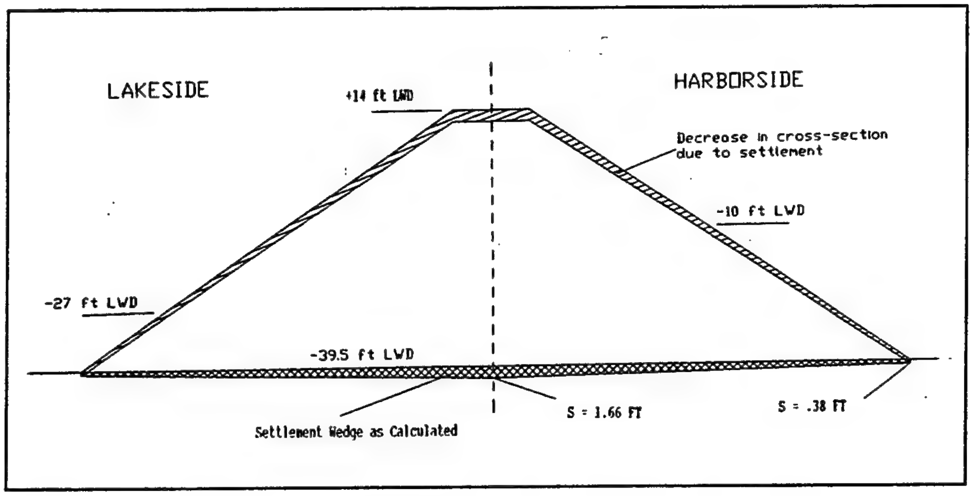

Figure 5-11. Expected cross-section adjustment due to settlement

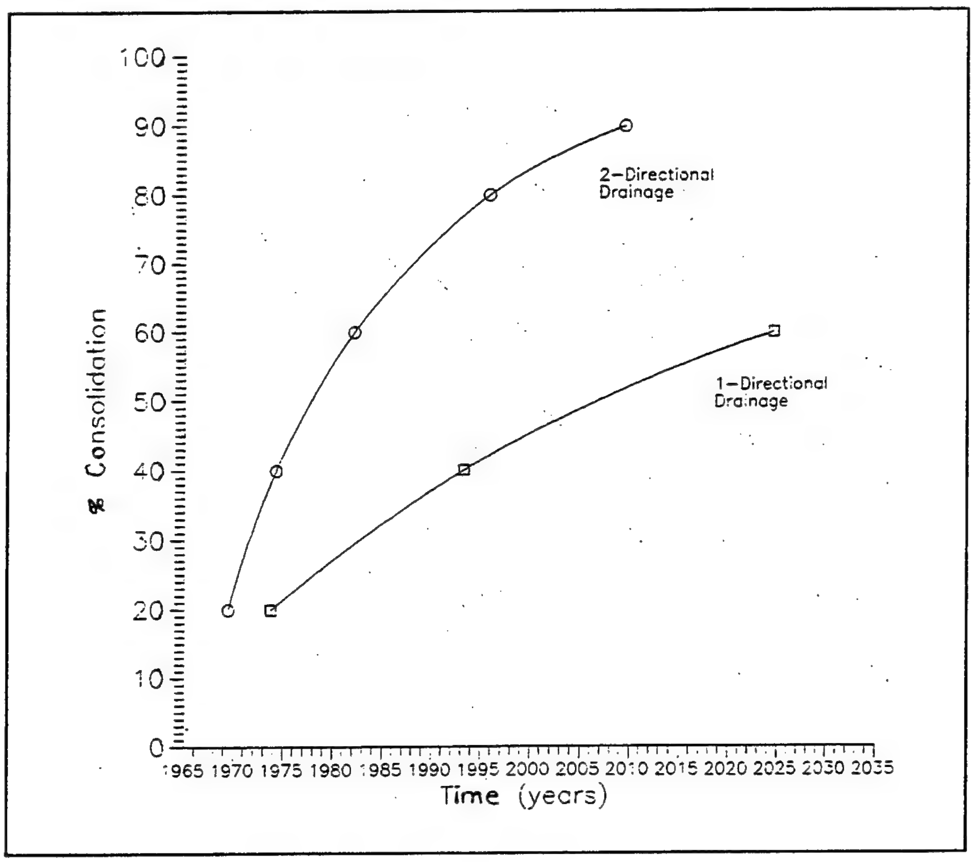

Figure 5-12. Calculated time rate of settlement

An investigation into construction practices during the breakwater's construction revealed that the foundation may not have been prepared as designed. For breakwater stations 43 through 60, located at Segments 7 and 8, excavation of the lake bed clay was conducted during October and November 1967. Excavated lakebed material (soft clay) was side-cast lakeward from point of removal. Sand backfill was not placed until April through June of the next year due to winter suspension of construction activities. Excavated or in situ lakebed materials may have been washed back into the excavated foundation area during the winter months. Construction of the breakwater on this foundation would cause larger than estimated settlement of the breakwater with eventual sinking or loss of stone in this material as discussed in earlier paragraphs.

Another measure of settlement using the three survey data sets is exhibited by the average crest elevation by segment for each year. Table 5-16 lists those crest elevations in addition to the change from the as-constructed crest elevation by segment.

Table 5-16
Average Crest Elevations (ft, LWD)

Segment No.	1967	1975	Difference	1989(92)	Difference
1	12.9	12.3	-0.6	---	---
2	12.5	12.5	0.0	12.0	-0.5
3	12.7	12.7	0.0	13.1	+0.4
4	12.7	13.5	+0.8	13.9	+1.2
5	12.8	13.4	+0.6	11.5	-1.3
6	13.3	13.6	+0.3	13.1	-0.2
7	13.3	13.0	-0.3	11.9	-1.4
8	13.2	11.9	-1.3	11.0	-2.2

Quantification of wave damages

Deterministic investigation of wave damages focuses on the stability of individual armor units (stones) under wave attack. Rubble-mound breakwaters are classified as statically stable structures. This implies that very little or no movement of the armor units is acceptable. Any displacement of stones is referred to as damage. Displacement of armor units to the toe of the breakwater (below -30-ft (-9-m) LWD) effectively removes the armor units from the functioning portion of the breakwater.

The armor unit layer of a rubble-mound breakwater is the only layer designed to resist the wave forces. Once the armor layer is lost, the underlayers are dispersed much more rapidly, which can lead to unraveling of

the entire structure. For this reason a loss of 10 percent of the armor stone is defined to be a severe damage condition. Ten percent of the armor layer in the case of Burns Harbor breakwater can be quantified as approximately 78 sq ft/ft (23.8 sq m/m).

Any attempt to quantify the damage for a given wave height above the design wave height produces a large variation in results, indicating that the wave damage process is very random. It has been shown that for regular-shaped stone, i.e. parallelpiped blocks, the damage versus wave height function will behave as a step-function relationship indicating various plateaus of stability in addition to the potential for severe progressive damage (Carver and Wright 1991).

Losada (1991) found that scattering in wave damage results are greater for breakwaters constructed with regular-shaped armor units, such as parallelepiped blocks, than for structures constructed with irregular-shaped armor units, such as tetrapods or quarry stones. A structure constructed of parallelepiped blocks has been found to be erratic in its damage patterns possibly due to extensive stacking of the armor units. For this reason, movement of one block in this type of structure can be followed by significant progressive failure of the armor layer.

It was observed during a stability investigation of rubble-mound structures subjected to extreme wave heights (Carver and Dubose 1986) that structures built to a 1V:1.5H slope generally tended to experience more damages than those constructed on a 1V:2H slope.

Calculation of predicted wave damages. Predicted armor layer damages were calculated using the revised average annual wave climate (Figure 5-10) and Table 5-17 (Table 7-9 in the *Shore Protection Manual* (1984)). This table is based on essentially the same Hudson stability tests used by the original designers of Burns Harbor breakwater to predict damages with some modifications which incorporated further rubble-mound stability testing.

Although current design guidance for the design of rubble-mound structures recommends the use of a design wave somewhere between $H_{1/3}$ and $H_{1/10}$, the rubble-mound stability testing upon which Table 5-17 is based utilized $H_{1/3}$ as the representative wave[1]. Therefore, to allow proper calculation of damages, the significant wave has been used in all calculations for this analysis.

[1] Personal Communication, Robert D. Carver, Hydraulic Engineer, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Table 5-17
Active Armor Unit Zone Damage as a Function of $H/H_D = 0$

Unit	Damage (D) in Percent						
	0 to 5	5 to 10	10 to 15	15 to 20	20 to 30	30 to 40	40 to 50
Quarrystone (smooth)	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarrystone (rough)	1.00	1.08	1.19	1.27	1.37	1.47	1.56
Tetrapod and Quadripods	1.00	1.09	1.17	1.24	1.32	1.41	1.50
Tribar	1.00	1.11	1.25	1.36	1.50	1.59	1.64
Dolos	1.00	1.10	1.14	1.17	1.20	1.24	1.27

H = Experienced wave height.
H_{D-0} = Design wave height.
(SPM 1984).

Expected damages were estimated using the design wave height of 15 ft (4.6 m) and a storm water level of +3.5 ft (1.1 m) LWD. Typically, Table 5-17 is used to address lakeside armor layer damages resulting from incident waves approaching perpendicularly to the structure. Figure 5-13 illustrates two areas of the cross section that are susceptible to the predicted damages.

The lakeside active armor unit zone extends from the center of the crest down to one design wave height below the water level on the lakeside. Thickness of the potential damage area is determined by the primary armor unit layers. Since the Burns Harbor breakwater experienced harborside damages also, the harborside armor layer was also used in the damage calculation. The harborside active armor unit zone was composed of one layer of armor to an elevation of -14 ft (-4.3-m) LWD and is also illustrated in Figure 5-13.

Segments 7 and 8 on the west arm of the breakwater were examined for the approximate percent of damages to be applied for the different structure alignment. The WIS database has shown that the predominant storm environment at Burns Harbor is generated out of the northwest. Taking the component of that wave direction which would be perpendicular to the west arm, a factor of 0.70 was determined. Since Segment 7 falls on the corner and would be especially vulnerable to diffracted wave energy, Segment 7 was treated in the same manner as the rest of the north arm of the breakwater. For Segment 8, however, a factor of 0.70 was applied to expected damages calculated using Table 5-17.

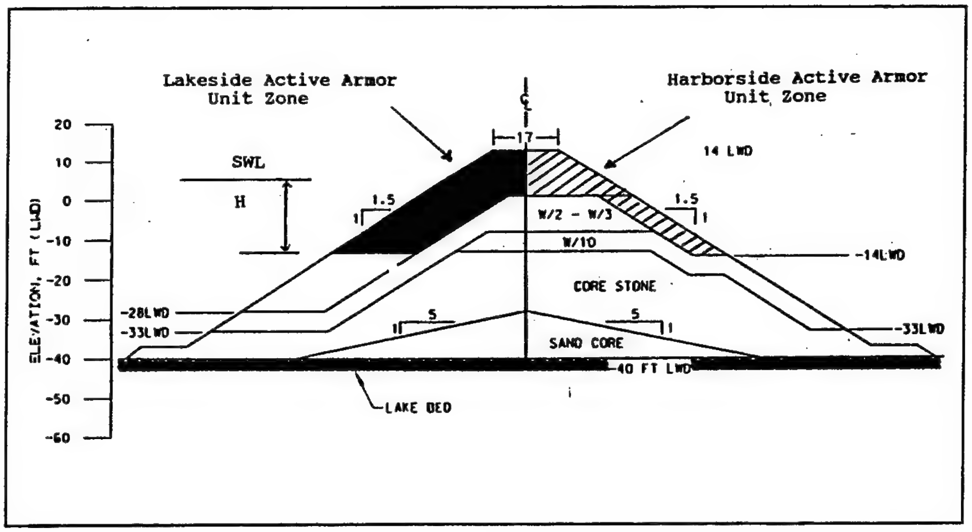

Figure 5-13. Active armor unit zone

Engineer Manual 1110-2-2904, "Design of Breakwaters and Jetties," (Headquarters, U.S. Army Corps of Engineers 1986) outlines a methodology to estimate average annual damages using the Hudson relationship. Defined ratios in the damage table are related to specific wave heights above the design wave height and their respective return intervals. Damages predicted by the probabilistic storm events are summed to result in average annual damages expected over the life of the structure. Table 5-18 illustrates the process used to determine mean average annual damages. This calculation of damages determined that under the average annual wave climate, 1.2 percent of the active armor unit zone can be expected to require replacement annually.

As previously discussed, empirical testing has shown that expected damages can be described by a range rather than a single expected value. To incorporate the damage range concept into the Hudson equation analysis, a potential variation in K_D was investigated. Based on the work of Carver and Wright (1991) and Losada (1991), the coefficient of variation of K_D for cut (regular-shaped) armor stone was estimated to be 37 percent; resulting in a standard deviation of 2.0, for a mean value of $K_D = 5.4$. In contrast, the standard deviation of K_D for angular armor stone, with the same mean value of K_D, would be expected to be less than 1.0.

Using the original K_D (1966) estimate of 5.4 and a standard deviation of 2.0, the potential range of K_D for the Burns Harbor structure would be 3.4 to 7.4. The corresponding zero damage wave height would then range from 12.8 to 16.6 ft (3.9 to 5.1 m). Since the below waterline active armor unit zone is a function of the zero damage wave height, the portion of the armor layer exposed to potential damage varies with the zero damage wave height. Applying the potential variability in the stability coefficient, the average annual percent damage to the active armor unit zone ranges from 0.1 to 9.6 percent.

Table 5-18
Determination of Mean Average Annual Damages

Ratio H/H_{D-O}	H ft	Return Interval yrs	Damage percent	Average Damage percent	Probability	Average Probability	Average Damage × Average Probability	Σ Probability
1.56	23.4	--	.45		.000			
				.40		.000	.000	.000
1.47	22.1	--	.35		.000			
				.30		.000	.000	.000
1.37	20.6	--	.25		.000			
				.21		.000	.000	.000
1.27	19.1	--	.175		.000			
				.15		.000	.000	.000
1.19	17.9	--	.125		.000			
				.10		.039	.004	.004
1.08	16.2	13	.075		.077			
				.05		.164	.008	.012
1.00	15.0	4	.025		.250			

H = experienced significant wave height.
H_{D-O} = design wave height.
Σ = summation of the percent damage to the active armor unit zone for the probabilistic return interval wave events.

Using the criteria given in Figure 5-13 and a 15-ft (4.6-m) design wave, the total active armor unit zone was calculated as 192,800 tons (175,000 mt), 114,000 tons (103,000 mt) on the lakeside and 78,800 tons (71,000 mt) on the harborside. Using 70 percent effective damages in Segment 8, the projected range in average annual damages is 186 tons (169 mt) to 17,800 tons (16,200 mt). All conversions from damage area to tons of stone used a porosity of 0.41 and a unit weight of 145 pcf (2,323 kg/m^3), which are characteristic of Bedford limestone.

Wave damages under climate experienced since construction. Estimating damages using the average annual wave climate is one way of calculating expected damages. Another method is to use the actual wave climate experienced, and then correlate it to the Hudson relationship to estimate damages over the life of a structure. The TMA-transformed WIS data set from 1965 through 1987 will be applicable for the time period since construction as opposed to the 50-year design lifetime period of the average annual wave climate estimate.

Stability calculations will use the significant wave heights, Hs, provided in Table 5-13. Seven storm events have exceeded the design wave height of

15 ft (4.6 m) over the life of the structure, documented through 1987. Again looking at the potential range in stability, for the design waves of 12.8 ft (3.9 m) and 16.6.ft (5.1 m), the number of storm events exceeding design were 18 and 1, respectively. Table 5-19 lists the mean damages expected using the storm events which exceeded the 15-ft (4.6-m) design wave height.

Table 5-19
Mean Damages Expected Under the Experienced Wave Climate

				Maintenance Stone Volume		
Storm Event	Wave Height ft	H/H_{D-0}	Damages percent	Lakeside tons	Harborside tons	Total tons
29 Jan 73	15.1	1.01	3.13	3,430	2,380	5,810
01 Feb 76	15.3	1.02	3.75	4,110	2,840	6,950
25 Dec 79	15.8	1.05	5.63	6,180	4,280	10,460
11 Nov 83	15.1	1.01	3.13	3,430	2,380	5,810
28 Feb 84	15.4	1.03	4.38	4,800	3,320	8,120
12 Feb 85	15.3	1.02	3.75	4,110	2,840	6,950
08 Feb 87	16.7	1.11	8.86	9,720	6,720	16,440
Total			32.63	35,780	24,760	60,540

The calculations in Table 5-19 indicate that for the breakwater subjected to the WIS-simulated wave climate, a potential mean damage estimate of 60,540 tons (54,920 mt) of stone would be estimated over the 21-year life of the structure. This estimate represents 1.6 percent of the active armor unit zone or 2,880 tons (2,600 mt) average annual maintenance. The range in potential damages represented by the variability of the stability coefficient is 0.15 to 8.5 percent of the active armor unit zone, 294 to 12,400 tons (267 to 11,250 mt) average annual maintenance.

Actual documented maintenance. Actual maintenance performed on the Burns Waterway Harbor breakwater over the life of the structure to date is shown in Table 5-20.

Total expenditures in 1993 dollars on the Burns Harbor breakwater are calculated at $8,498,347, which results in an average annual maintenance cost of $404,683 over a 21-year period. Expenditures for maintenance surveys were retained in the total cost as an element of annual maintenance. By dividing the total tonnage of maintenance stone (145,117 tons (131,648 mt)) into the total expenditures over the life of the structure, a cost of $58.6/ton (65.11/mt) in 1993 dollars was calculated. This value was used in conjunction with the damage estimates to calculate the potential distribution of damages.

Table 5-20
Annual Maintenance Expenditures

Year	Total Actual Expenditure dollars	ENR (a) Construction Price Index	Constant 1993 Dollar Expenditure dollars
1973	11,509	2.6611	30,626
1974	8,618	2.4999	21,544
1975	185,000	2.2835	422,444
1976	1,122,532	2.1051	2,363,052
1977	215,459	1.9583	421,934
1978	405,694	1.8206	738,604
1979	40,003	1.6815	67,263
1980	981,532	1.5602	1,531,374
1981	463,466	1.4285	662,063
1982	300,623	1.3199	396,794
1983	617	1.2405	765
1984	0	1.2188	0
1985	62,336	1.2065	75,209
1986	172,520	1.1754	202,772
1987	58,181	1.1458	66,665
1988	190,357	1.1167	212,574
1989	1,172,824	1.0954	1,284,664

Source: Report of the Chief of Engineers, 1973-1989.
(a) Base Year = 1993.

Table 5-21 lists the various wave damage predictions in terms of average annual maintenance costs as well as the range in values associated with the potential variation in K_D.

Figure 5-14 illustrates the potential distribution of damages for the two damage modes examined. As can be seen, the degree of variability in the predicted wave damages is higher than in the predicted settlement damages. Another factor affecting the correlation of maintenance and perceived damages to damage mode involves the magnitude of damage. It is not clear how much of a gradual 0.5- to 1.5-ft (0.2- to 0.5-m) settlement reduction in cross section would be addressed by maintenance. Damages due to waves and slope failures are more likely to generate maintenance activities.

Rubble-mound stability investigations, conducted since the original breakwater design, have identified at least one other design issue which may

Table 5-21
Estimated Average Annual Maintenance Costs Due to Waves

Method	Design Life Estimate years	Percent of Active Armor Unit Zone	Average Annual Cost 1993 dollars
Original design (H_D = 15 ft)	50	0.0	0
Average annual wave climate (H_D = 15 ft)	50	1.2	$130,700
Experienced wave climate (H_D = 15 ft)	21	1.6	$169,700
Actual documented maintenance	21	3.7	$404,700
Hudson lower limit (H_D = 16.6 ft, K_D = 7.4)			
• Average annual wave climate	50	0.1	$10,900
• Experienced wave climate	21	0.15	$17,200
Hudson upper limit (H_D = 12.8 ft, K_D = 3.4)			
• Average Annual Wave Climate	50	9.6	$1,043,000
• Experienced Wave Climate	21	8.5	$727,000

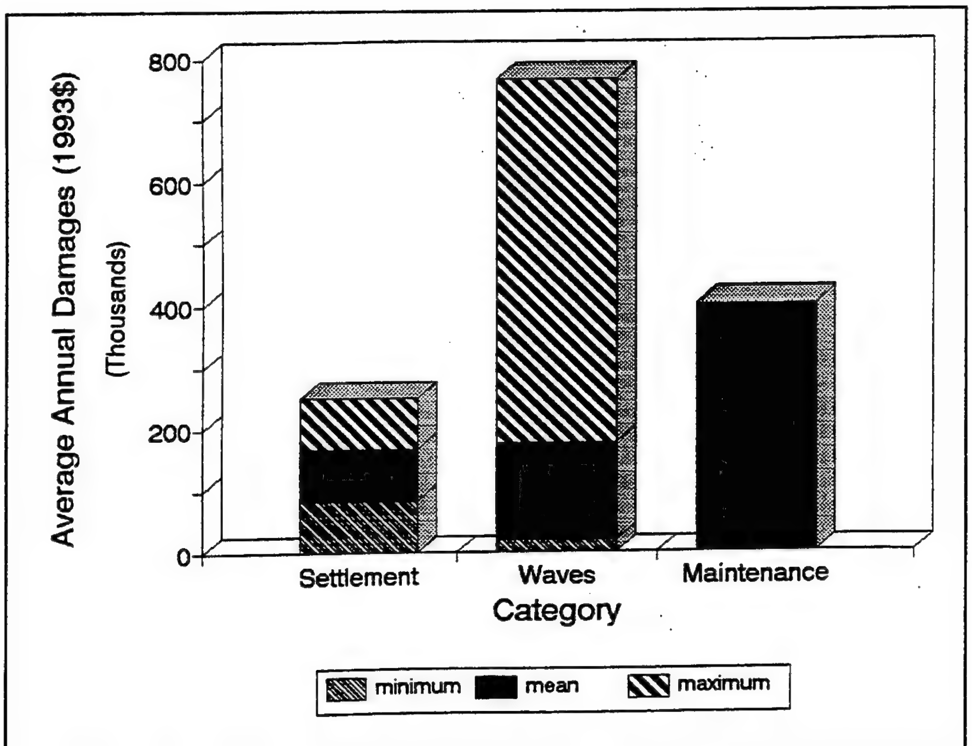

Figure 5-14. Theoretical range of damages

affect stability at Burns Harbor breakwater. Walker, Palmer, and Dunham (WPD 1975) examined the backslope (or leeside) stability of breakwaters.

Backslope stability of a breakwater. The Hudson equation as well as the majority of rubble-mound stability theories do not address the leeside stability of a rubble-mound structure. A study done by WPD 1975 examined the backslope stability of a breakwater in terms of incident wave height, crest elevation, water elevation, and armor unit weight on the front and backsides. Figure 5-15 illustrates their preliminary results.

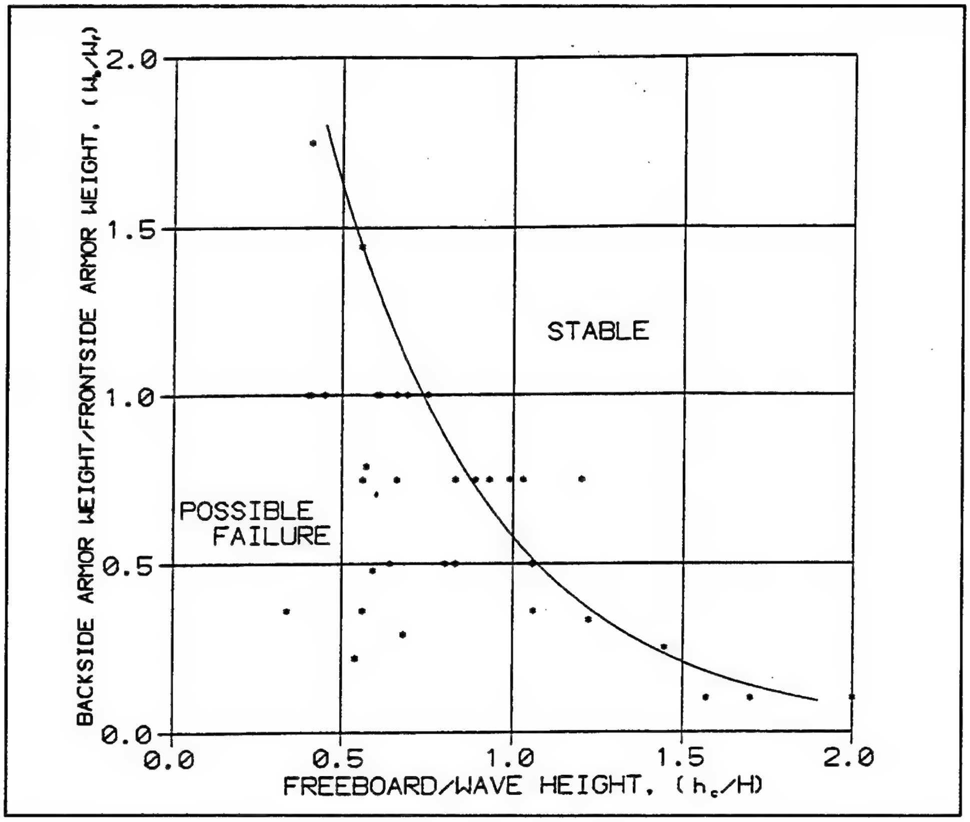

Figure 5-15. Backslope stability relationship

The empirical WPD 1975 equation, representing a stability threshold, is given below.

$$\frac{W_b}{W_f} = EXP\ [(-C_1)(h_c/H)]\ C_2 \tag{5-1}$$

where

W_b = mean stone weight on backside of breakwater

W_f = mean stone weight on frontside of breakwater

C_1 = 2.061

C_2 = 4.554

h_c = breakwater freeboard (crest elevation above still-water level)

H = incident wave height

Note that the Equation 5-1 is extremely sensitive to breakwater crest elevation (freeboard). The higher the crest, the more stable the harborside of the breakwater; even if a smaller armor stone is used on the backside of the breakwater. The most important parameter in establishing the crest elevation is the design wave height. Table 5-22 lists the application of this empirical equation to the storm events at Burns Harbor breakwater.

As can be seen from the application of the backslope stability relationship to the Burns Harbor structure, certain combinations of water level and wave height can result in a vulnerable harborside situation. Nine documented storm events were classified as possible failure situations over the life of the structure. When compared to the mean number of damaging storm events determined using the Hudson relationship, this analysis indicates that at least two more storms may have caused damage to the harborside than the lakeside.

The average storm water level for all possible damaging events was 4.09 ft (1.2 m) LWD, ranging from 2.93 to 4.85 ft (0.9 to 1.5 m). A water level of 4.09 ft (1.2 m) leaves approximately 9.9. ft (3 m) of freeboard for the breakwater. The average significant wave height for those events was 15.1 ft (4.6 m), ranging from 13.2 to 16.7 ft (4 to 5.1 m). Using the February 1987 storm event as a design storm, the crest elevation would need to be set at +17 ft (+5.2 m) LWD, or at least 3 ft (0.9 m) higher than the design crest elevation of +14 ft (+4.3 m) LWD to prevent backslope instability.

Comparison of the results from Equation 5-1 to model study results of the current breakwater configuration (Carver, Dubose, and Wright 1993) has shown favorable agreement between predicted harborside damages and resultant breakwater damage experienced in the model study. Furthermore, the maintenance history of Burns Harbor breakwater indicates that the breakwater's harborside was damaged as frequently as the lakeside.

The inclusion of the harborside active armor unit zone in the damage calculations should incorporate some of the backslope instability. In the case of Burns Harbor breakwater, the harborside armor is placed in a one-layer thickness. Significant damages caused by overtopping could remove the armor layer and damage underlayers below. The method of calculating the appropriate damage area on the harborside due to overtopping has not been defined.

Table 5-22
Application of Backslope Stability Relationship to Burns Harbor Breakwater

Date of Storm	Wave Height (H) ft	SWL ft, LWD	Hc = 14 - SWL ft, LWD	Hc/H	Expected Structure Stability During Storm[1]	Theoretical Stone Size Needed[2] on Leeward Slope
30 Nov 71	12.6	2.78	11.22	0.890	Stable	9.4
14 Nov 72	12.4	4.54	9.46	0.763	Stable	12.3
29 Jan 73	15.1	4.29	9.71	0.643	Possible Failure	15.7
15 Feb 73	12.9	4.17	9.83	0.762	Stable	12.3
18 Mar 73	13.2	4.35	9.65	0.731	Possible Failure	13.1
22 Feb 74	13.9	4.35	9.65	0.694	Possible Failure	14.2
13 Nov 75	12.6	3.72	10.28	0.816	Stable	11.0
01 Feb 76	15.3	2.93	11.07	0.724	Possible Failure	13.3
22 Feb 76	14.2	3.44	10.56	0.744	Stable	12.8
20 Dec 76	13.1	2.25	11.75	0.897	Stable	9.3
14 Jan 79	13.4	2.30	11.70	0.873	Stable	9.8
26 Feb 79	12.9	2.57	11.43	0.886	Stable	9.5
25 Dec 79	15.8	3.68	10.32	0.653	Possible Failure	15.4
26 Feb 80	13.0	3.03	10.97	0.844	Stable	10.4
02 Dec 80	13.1	3.19	10.81	0.825	Stable	10.8
24 Dec 80	13.5	2.79	11.21	0.830	Stable	10.7
20 Nov 81	12.6	3.08	10.92	0.867	Stable	9.9
11 Nov 83	15.1	4.01	9.99	0.662	Possible Failure	15.1
16 Nov 83	13.4	3.25	10.75	0.802	Stable	11.3
28 Feb 84	15.4	4.02	9.98	0.648	Possible Failure	15.6
12 Feb 85	15.3	4.35	9.65	0.631	Possible Failure	16.1
08 Feb 87	16.7	4.85	9.15	0.548	Possible Failure	19.1

[1] Stability determination based on Wb/Wf = 1.0; structure stable if Hc/H > 0.74
[2] Stone size on lakeward face assumed to be Wf = 13 tons

Total actual changes compared to total expected changes

Actual changes in the breakwater cross-sectional area were determined for each breakwater segment by calculating the average change in area between survey-generated cross sections over a given time interval. Total mean expected changes were determined by adding the positive area change associated with the maintenance stone to the negative area change due to the mean estimated wave and settlement damages over a given segment. The residual, as defined in this analysis, was actual changes minus the expected changes. An example of actual and expected area change estimates is given algebraically below.

Actual area change (1975-1989) = average cross-sectional area (1989) minus the average cross-sectional area (1975)

a. positive value indicates cross-sectional area gain.
b. negative value indicates cross-sectional area loss.

Expected area change (1975-1989) = maintenance stone placed minus (mean wave damage plus mean settlement damage)

a. positive value indicates maintenance greater than damage.
b. negative value indicates damage greater than maintenance.

Residual = actual area change minus the expected area change

a. positive value indicates "less than expected" damages.
b. negative value indicates "greater than expected" damages.
c. large residual, positive or negative, implies a departure from theoretical calculations.

To determine if the expected changes correctly predicted the measured area changes, a mass balance approach was used. The entire cross section was analyzed from +14 ft (4.3 m) LWD to -30 ft (-9.1 m) LWD for total changes. Complete surveys of the breakwater were conducted in 1967 (as-built), 1975, and 1989. Therefore, in addition to the total time period from 1967 through 1989, two incremental time periods were available for analysis, 1967 through 1975 and 1975 through 1989.

To make a statistical analysis of the cross-sectional area of the breakwater possible, it was necessary to establish a lower elevation limit for area measurement which was applicable to all cross sections. Due to the varying lake bed elevation, a lower limit of -30 ft (-9.1 m) LWD was established. Actual lakebed elevations as taken from the 1989 survey vary along the structure from -30 ft (-9.1 m) LWD to -41 ft (-12.5 m) LWD, respectively, from Segment 8 to Segment 1. The omission of the cross-sectional area below -30 ft (-9.1 m) LWD represents about 10 percent of the breakwater cross-sectional area.

The comparison of overall changes, therefore, addresses the area from +14 ft (+4.3 m) LWD to -30 ft (-9.1 m) LWD. This area will include all the damages due to wave action and the majority of the damages due to settlement. Armor units which fall outside this area do not contribute to the condition or the design performance of the breakwater.

The time frames of analysis represent different phases of breakwater maintenance and severity of damage. From 1967 to 1975 no maintenance was performed on the breakwater. The storm history indicates that approximately 10 percent of the total wave damages should have occurred during this time span. The settlement investigation indicated, however, that approximately 40 percent of the settlement would have occurred by 1975.

From 1975 to 1989, 145,100 tons (131,720 mt) of maintenance stone was placed on the breakwater. The storm history indicates that this time span was much more damaging, with 90 percent of the damage due to waves. Approximately 60 percent of the settlement is believed to have occurred between 1975 and 1989.

The time frame from 1967 through 1989 was used to evaluate the full life of the structure, including total mean damages and total maintenance. Figure 5-16 illustrates the cross-sectional area change over the structure's life in terms of lower and upper area changes. These values were taken from comparisons of surveyed cross sections and examine the first performance issue of actual changes to the structure.

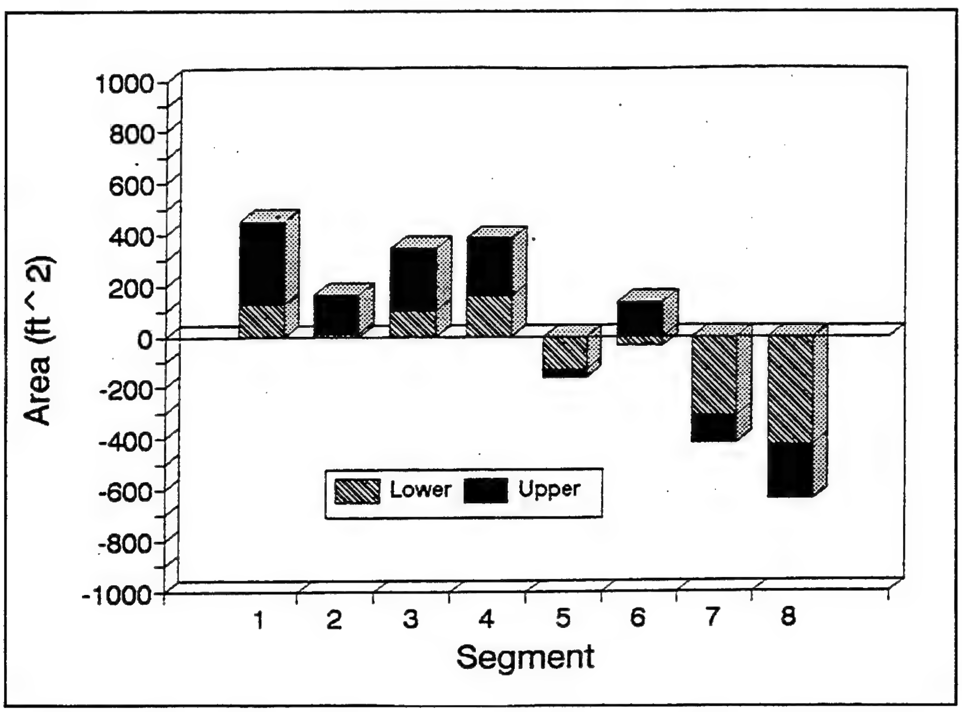

Figure 5-16. Cross-sectional area change of breakwater

Segments 1, 2, 3, and 4 show a net increase in cross-sectional area from the as-built survey for both the upper and lower regions indicating relatively good present condition. Segment 6 shows an increase in area for the upper region and a relatively minor decrease in area for the lower region. Segments 5, 7, and 8 show a significant decrease in cross-sectional area for both the upper and lower regions with Segments 7 and 8 being the most critical. Segments 7 and 8 were scheduled for maintenance in the 1994/1995 construction seasons.

Segments 5, 7, and 8 have received the least amount of maintenance over the life of the structure, decreasing respectively with segment. Segment 8 has never been maintained during the life of the structure which is reflected in the significantly poorer condition of the present cross section. Segment 7 has received minimal maintenance.

Figure 5-17 illustrates the range in potential damages as calculated from the settlement and wave damage analyses for each segment. The high degree of variability is largely attributable to the randomness of the wave damages. Figure 5-18 illustrates the mean predicted damages due to both settlement and wave damages and the maintenance stone placed per segment. The upper and lower limits of damages indicate the potential range of total damages.

Table 5-6 summarizes the confidence intervals assumed for the survey data comparison. Condition surveys have a 1-ft (0.3-m) vertical accuracy. This is the same as the construction tolerance evaluated in Table 5-6. Therefore, statistically significant differences occur for area differences greater than 158 sq ft (14.7 sq m).

Actual and expected changes. Two types of graphs were used to evaluate breakwater design performance (Figures 5-19, 5-20, and 5-21). The first issue, actual observed changes in the breakwater, is illustrated on the first type of graph as actual changes or measured changes between survey years. This type of graph is an "area change" graph.

The second issue, the ability to explain breakwater changes, is illustrated in the first graph in the comparison of actual and mean expected changes. It is also shown in the second graph as the residual of actual changes minus the mean expected changes. The second graph is a "residual" graph.

The "area change" graph compares the mean expected change in area to the actual change in area for the cross section. The percent change in actual area from the template area is illustrated through the scale on the right of the graph. The template area is 3650 sq ft (339 sq m). Tables 5-23 and 5-24 list actual and mean expected changes for the three time periods.

The "residual" graph displays the residual of area change represented by the actual change in area minus the mean expected change in area. Essentially this type of graph quantifies the accuracy of the mean theoretical prediction of damage or change. The dashed lines on each graph indicate the detection threshold of the comparison which is actually the potential survey error

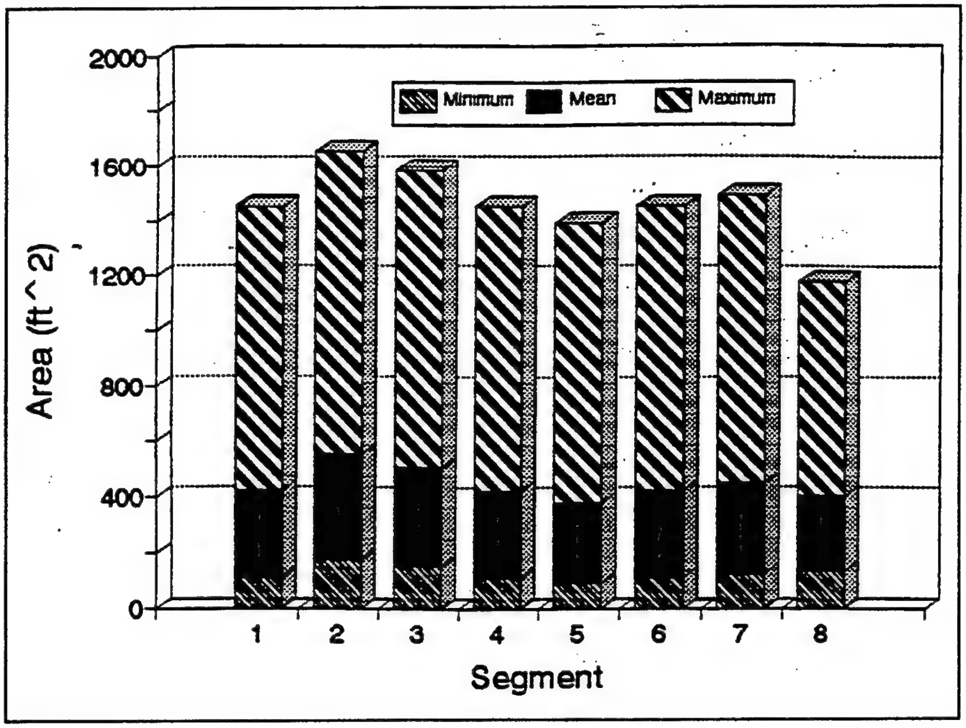

Figure 5-17. Potential damages, wave and settlement

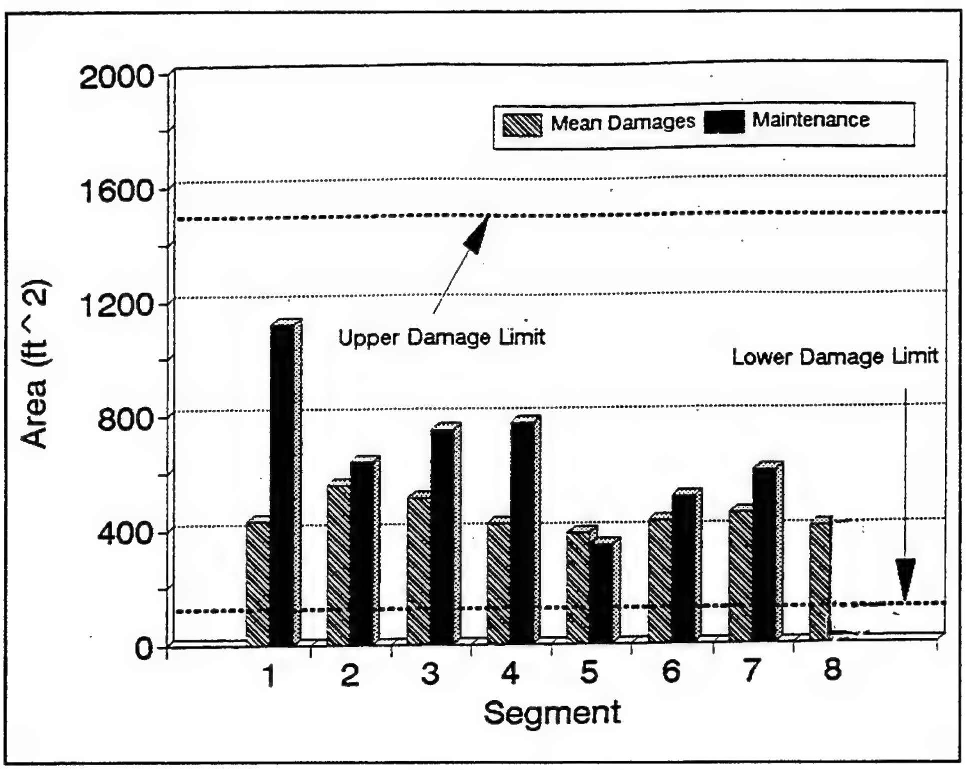

Figure 5-18. Mean damages and maintenance

Table 5-23
Actual Changes in Cross-Sectional Area

Breakwater Segment	1967-1975 sq ft	1975-1989 sq ft	1967-1989 sq ft
1	-22	+476	+454
2	-95	+258	+163
3	-28	+376	+348
4	+9	+373	+382
5	+84	-257	-173
6	+34	+63	+97
7	-182	-308	-490
8	-157	-508	-665

Table 5-24
Mean Expected Changes in Cross-Sectional Area

Breakwater Segment	1967-1975 sq ft	1975-1989 sq ft	1967-1989 sq ft
1	-95	+784	+689
2	-147	+226	+78
3	-128	+361	+232
4	-92	+439	+347
5	-76	+36	-41
6	-92	+184	+91
7	-104	-74	-179
8	-109	-298	-407

calculated as an area. Values which are less than this detection threshold are considered to be within the survey error and are not statistically significant.

Damages which are "less than expected" are illustrated as positive residuals while damages which are "greater than expected" are shown as negative residuals. High residual values (positive or negative) indicate a large discrepancy between mean theoretical and measured damages. A high positive residual indicates that the structure performed better than expected. If the residuals for all the segments were high positive residuals, it would indicate that the structure has been overdesigned. A high negative residual indicates that the structure is either experiencing unexpected damages in that segment or the experienced damages are above the mean estimated damages. Table 5-25 lists residual differences for the cross-sectional area comparisons. For the conversion of 1989 repair stone volumes to area change, values of 175 pcf (2,803 kg/m^3) specific weight and 0.30 void ratio were used. All other

Table 5-25 Residual Changes in Cross-Sectional Area			
Breakwater Segment	1967-1975 sq ft	1975-1989 sq ft	1967-1989 sq ft
1	+73	-308	-235
2	+52	+32	+85
3	+100	+15	+116
4	+101	-66	+35
5	+160	-293	-132
6	+126	-121	+6
7	-78	-234	-311
8	-48	-210	-258

maintenance stone to area conversions used 145 pcf (2,323 kg/m^3) specific weight and 0.41 void ratio.

Comparison of cross-sectional area for 1967-1975. The comparison of mean expected to actual changes for 1967 through 1975 is shown in Figure 5-19. The top graph plots actual and mean expected changes to cross-sectional area by segment. The bottom graph plots the residual or the actual change minus the mean expected change. Table 5-26 summarizes values used in this analysis. No maintenance was placed during this time period. Expected changes are a combination of approximately 40 percent of the total settlement and 10 percent of the total wave damages.

Actual changes illustrate changes during this time period of less than 10 percent of the template area. These relatively small changes mirror the minor damages which were expected to occur. Segments 7 and 8 exhibited the largest negative area changes and were at or above the detection threshold.

Actual and mean expected changes for all segments are less than or very close to the detection threshold dictated by the potential survey error. None of the residuals indicate significantly greater than mean expected damages. Due to all the actual as well as the expected values falling within the range of the detection threshold, it is difficult to draw further conclusions.

Comparison of cross-sectional area for 1975-1989. Mean expected changes are compared to actual changes for 1975 through 1989 in Figure 5-20. Table 5-27 summarizes values used for this time frame.

Mean expected changes for this time period are a combination of approximately 60 percent of the total settlement and 90 percent of the total wave damages.

All segments show actual positive area changes during this time period except Segments 5, 7, and 8. The largest negative change in area was

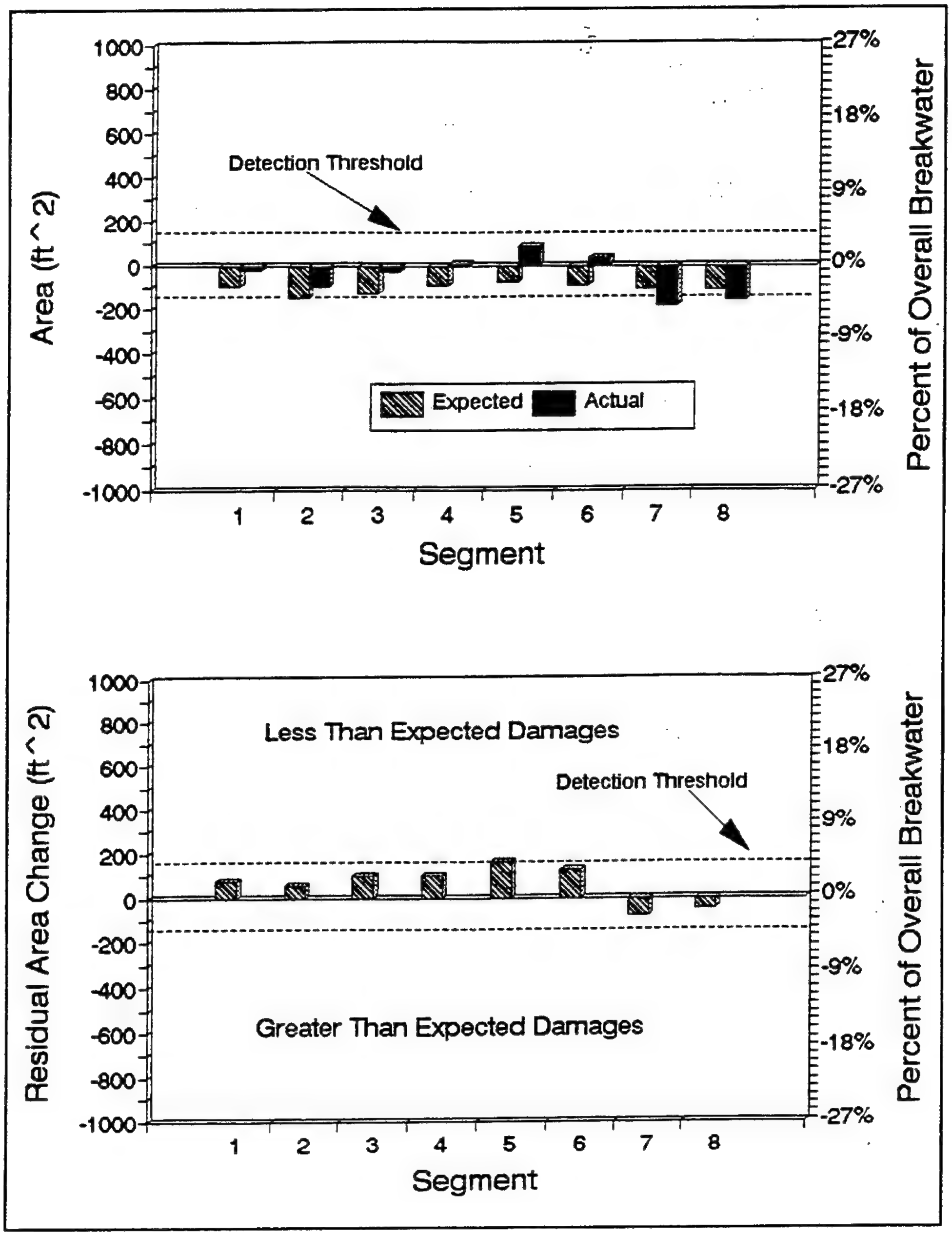

Figure 5-19. Area changes and residuals for 1967-1975

Segment 8 with a -14 percent change. Expected trends simulate the actual changes except for Segment 5, which should have exhibited a positive change due to maintenance efforts.

In magnitude, all segments except 2 and 3 exhibited greater than expected damages. That is, actual negative changes to the breakwater were more negative than those predicted by theory under the given environment and actual positive changes were not as great as they should have been, given the maintenance performed. Of those segments, however, four (Segments 1, 5, 7, and 8) were significantly beyond the detection threshold.

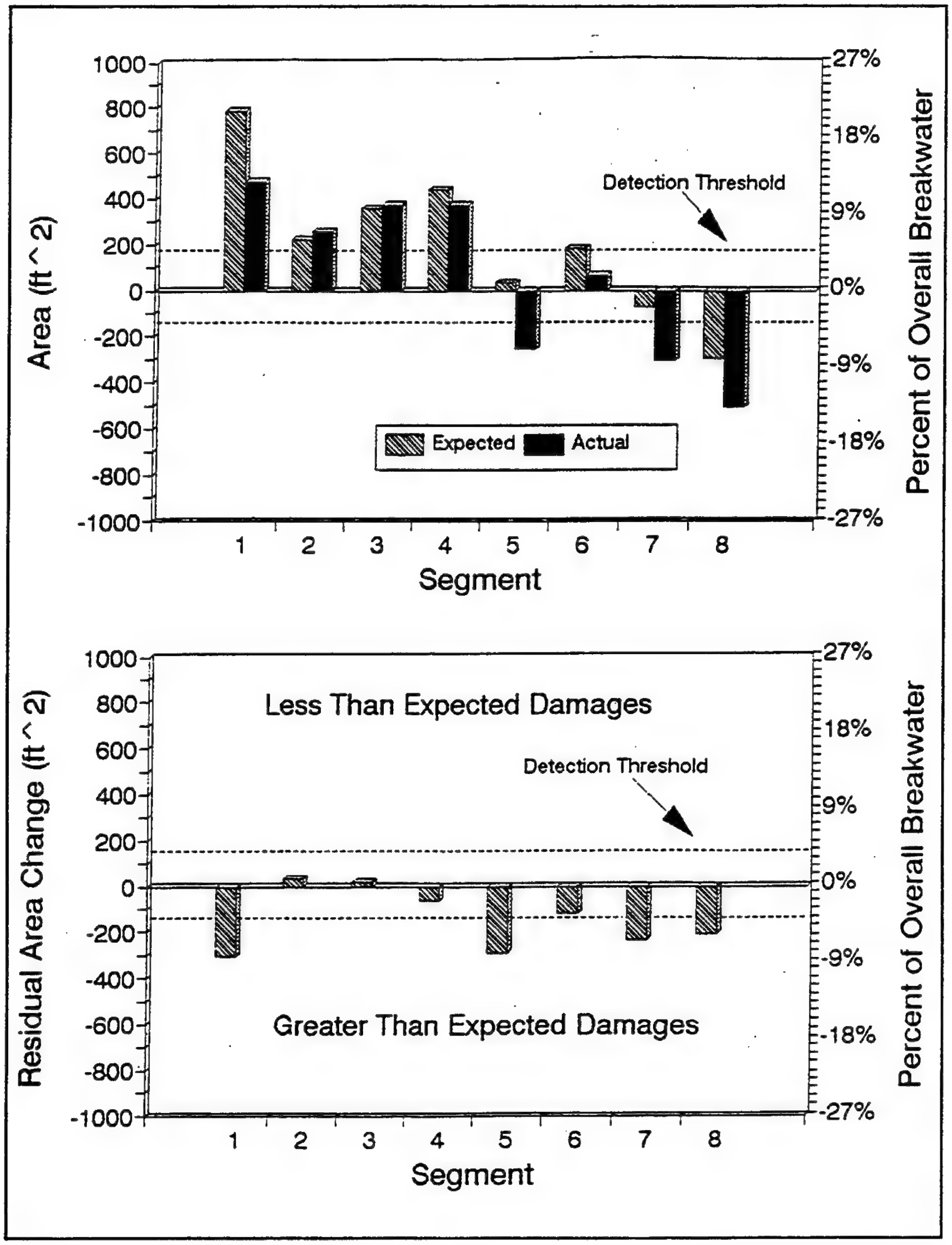

Figure 5-20. Area changes and residuals for 1975-1989

Table 5-26
Mass Balance Values for the Time Period 1967-1975

Segment	Area Change[1] sq ft	Maintenance sq ft	Settlement sq ft	Wave Damage sq ft
1	-22	0	-69	-26
2	-95	0	-121	-26
3	-28	0	-102	-26
4	+9	0	-66	-26
5	+84	0	-50	-26
6	+34	0	-66	-26
7	-182	0	-78	-26
8	-157	0	-91	-18

[1] Values greater than 158 sq ft are statistically significant.

Table 5-27
Mass Balance Values for the Time Period 1975-1989

Segment	Area Change[1] sq ft	Maintenance sq ft	Settlement sq ft	Wave Damage sq ft
1	+476	+1118	-104	-230
2	+258	+638	-182	-230
3	+376	+745	-154	-230
4	+373	+769	-100	-230
5	-257	+342	-76	-230
6	+63	+514	-100	-230
7	-308	+274	-118	-230
8	-508	0	-137	-161

[1] Values greater than 158 sq ft are statistically significant.

Damages which are "greater than expected" can result from two separate scenarios: (a) a significant maintenance effort was conducted and the placement of stone cannot be accounted for in the cross-sectional area above -30 ft (-9.1 m) LWD, or (b) actual area loss is greater than expected area loss.

Comparison of cross-sectional area for 1967-1989. Mean expected changes are compared to actual changes for 1967 through 1989 in Figure 5-21. Table 5-28 lists values used for this time period. Comparisons for the overall time period should be the most conclusive of all the time periods since all of the information was utilized.

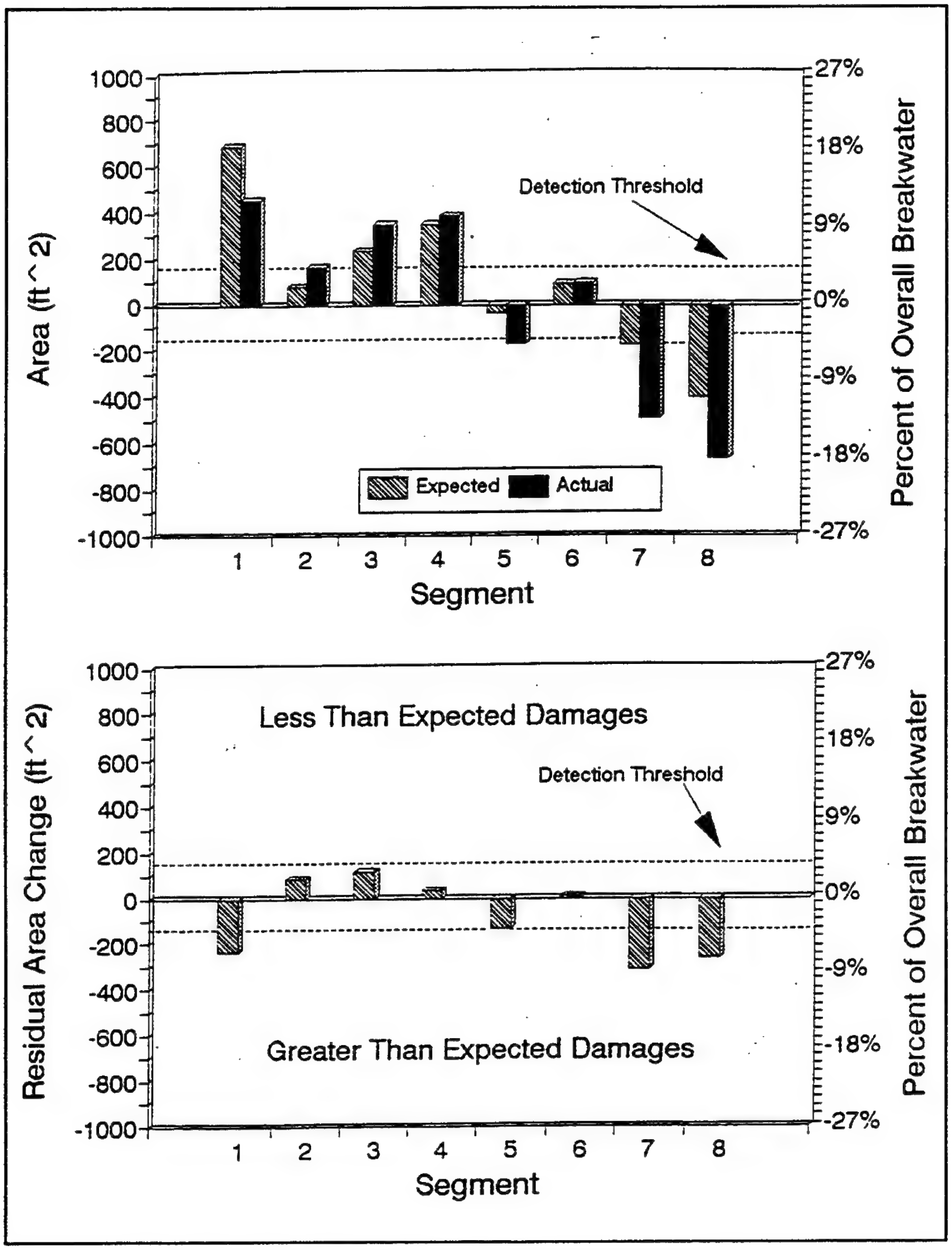

Figure 5-21. Area changes and residuals for 1967-1989

In terms of actual changes, five of the eight segments show a net positive area change ranging from +3 to +12 percent due to maintenance over the life of the structure. Segments 5, 7, and 8, however, show a decrease in area ranging from -5 to -18 percent of the cross-sectional area.

Mean expected changes followed the same trend of change for all segments. In magnitude, no segments had significantly less than expected damages, while four segments (Segments 1, 5, 7, and 8) showed greater than expected damages. Segment 7 exhibits the most critical response over the life of the structure.

Table 5-28
Mass Balance Values for the Time Period 1967-1989

Segment	Area Change[1] sq ft	Maintenance sq ft	Settlement sq ft	Wave Damage sq ft
1	+454	+1118	-173	-256
2	+163	+638	-304	-256
3	+348	+745	-257	-256
4	+382	+769	-166	-256
5	-173	+342	-127	-256
6	+97	+514	-167	-256
7	-490	+274	-197	-256
8	-665	0	-228	-179

[1] Values greater than 158 sq ft are statistically significant.

Discussion of cross-sectional mass balance analysis. Two general issues were examined in this analysis of the Burns Harbor breakwater. The first issue dealt with identifying the location and magnitude of cross-sectional area change occurring over the life cycle of the structure. Positive changes in cross-sectional area due to maintenance were compared to area loss. Temporal aspects of breakwater cross-sectional changes were used to identify specific problem areas. This was done by comparing three survey years; 1967, 1975, and 1989.

The second mass balance issue dealt with the assessment of how well the mean expected changes matched the actual changes; that is, how well state-of-the-art theory predicts prototype behavior. This is important for future design efforts as well as future predictions for maintenance in terms of quantity and cross-sectional location. Whether a theoretical prediction is found to be conservative or not will affect the accuracy of future maintenance estimates.

Two general damage modes were examined. Those were damages due to waves and settlement. For the three time periods analyzed, expected distributions of damages for each of the damage modes were predicted. For waves, 10 percent of the damages were expected to occur from 1967 to 1975 and 90 percent of the damages were expected to occur from 1975 to 1989. For settlement, 40 percent of the damages were expected to occur from 1967 to 1975 and 60 percent of the damages were expected from 1975 to 1989.

Location and magnitude of cross-sectional area change was calculated from the difference in area between time of construction and 1989(92). This value illustrated that while five of the eight breakwater segments increased in size due to maintenance activity, Segments 5, 7, and 8 decreased in size both in the upper and the lower regions.

Another measurement of location and magnitude of area change was obtained from individual time periods of breakwater history. Time periods used were 1967 to 1975 and 1975 to 1989. The difference between mean expected and actual damages was calculated. This resulted in a distinction between greater than or less than expected damages.

For the first time period, 1967 to 1975, actual breakwater changes were minor. Due to the mild storm climate for this time period, wave damages were expected to be only 10 percent of the total experienced over the structure life. Settlement damages were expected to be 40 percent of the total. Actual and expected changes, in general, were shown to be within the detection threshold. Segments 7 and 8 exhibited the largest negative area change, -5 percent of the template.

For the second time period, 1975 to 1989, more significant changes were exhibited both in terms of damage as well as maintenance activity. Damages expected for this time period were 90 percent of the total wave damages and 60 percent of the total settlement damages. Except for Segments 2 and 3, all segments exhibited greater than expected damages. Of those segments, however, four segments (1, 5, 7, and 8) were significantly beyond the detection threshold.

The final time period examined, 1967 to 1989, is the most comprehensive, since it incorporates all of the information available. Area losses for Segments 5, 7, and 8 ranged from -5 to -18 percent of the template. The residuals for Segments 1, 5, 7, and 8 indicate underestimates of the actual damages using state-of-the-art techniques.

The two main performance issues can be illustrated in graphical form. The first issue of location and magnitude of cross-sectional area change is somewhat difficult to measure because maintenance activity also changes the cross-sectional area. An absolute measure of change can be achieved through the damage magnitude measured in percent of breakwater template area. This value is the difference between the maintenance performed and the area change since construction.

If the cross-sectional area change is positive, then the damage magnitude will represent only that maintenance which went toward addressing damages. If the cross-sectional area change was negative, the damage magnitude includes all of the maintenance placed to address damages plus the existing deficit in cross-sectional area.

The second issue of how well the mean expected changes matched the actual changes is illustrated using the residual. Residuals represent actual changes minus mean expected changes. This value quantifies the accuracy of the theoretical prediction of damage or change. Positive residuals indicate less than expected damages while negative residuals indicate greater than expected damages. For greater than expected damages, either another failure mechanism was involved at these locations or the degree of failure was underestimated indicating a less than conservative design. Table 5-29 lists the values described above. All values are given in percent of overall template.

Table 5-29
Breakwater Performance Parameters

Breakwater Segment	Area Change percent	Maintenance percent	Damage Magnitude percent	Residual percent
1	+12	+31	19	-6
2	+5	+17	12	+2
3	+10	+20	10	+3
4	+11	+21	10	+1
5	-5	+9	14	-4
6	+3	+14	11	0
7	-12	+8	21	-9
8	-18	0	18	-7

Figure 5-22 illustrates these values in a graphical form. The cross-sectional area change since construction has followed the same general trend as the maintenance since construction along the structure length, indicating a positive correlation between maintenance activity and area change, as would be expected. The maximum area change, however, is only +12 percent of the overall template and three segments exhibit a negative change. Negative area changes along the west end of the structure reflect decreasing maintenance activity for this reach. Maintenance efforts were greatest on Segment 1 decreasing generally in magnitude in the westerly direction.

Figure 5-23 plots the damage magnitude against the residual. Looking at the segments individually, those segments with a high damage magnitude (in percent of overall template) and a negative residual are the most critical. Segments 1, 5, 7, and 8 clearly fall into that category. The other segments, while they may have experienced a higher damage magnitude than was expected at the time of the design, have exhibited predictable patterns for both wave and settlement damages.

Taking the weighted average (according to segment length) of damage magnitude, it was found that over the life of the breakwater, there was a 14-percent damage magnitude. This means that 14 percent of the overall breakwater template required replacement due to damages.

Conclusions

MCCP investigation into the original design of the breakwater determined that the design parameters of wind speed, water level, and fetch length were underestimated, resulting in an underestimation of the design wave. The design wave of 15 ft (4.6 m) used for the rubble-mound stability was found to be less that a 5-year return interval event. The design wave of 11 ft (3.4 m) used for the crest elevation design was found to be less than a 1-year return interval event.

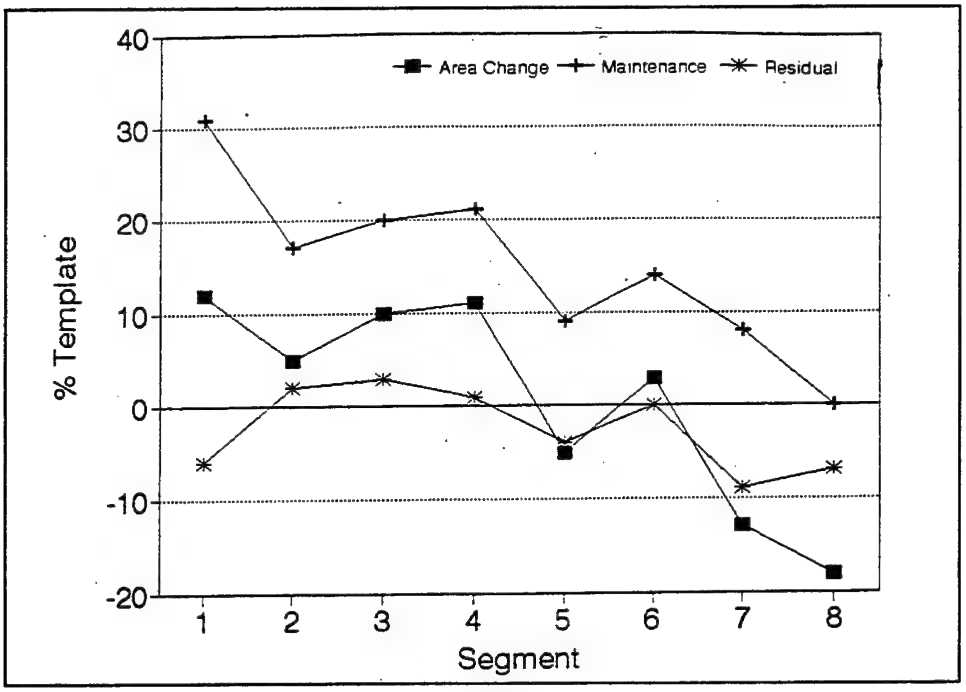

Figure 5-22. Performance evaluation; area change, maintenance, and residual

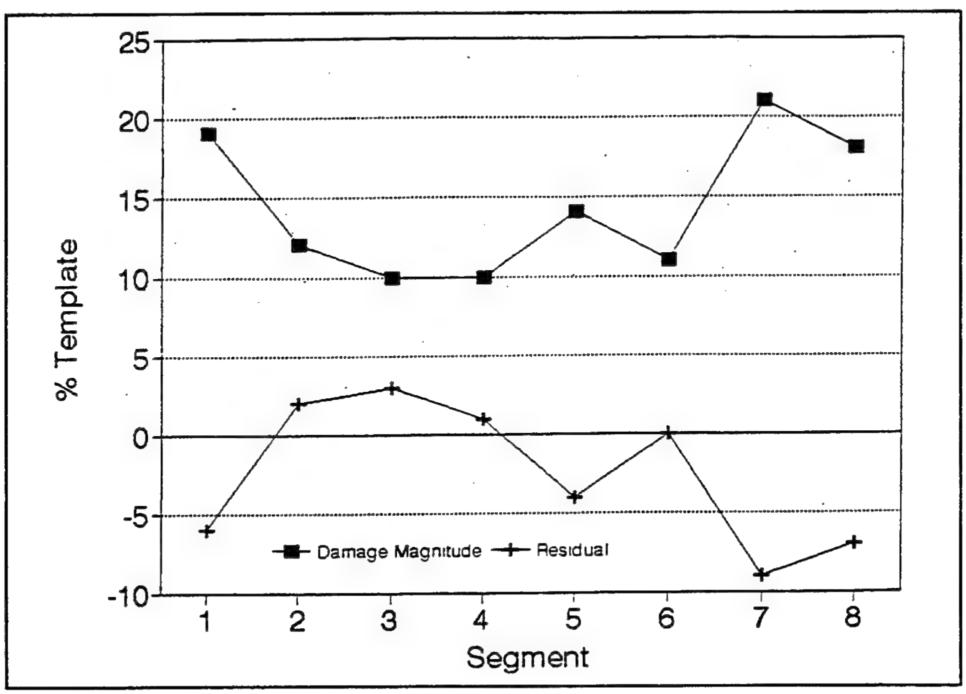

Figure 5-23. Performance evaluation; damage magnitude and residual

In this study, a rubble-mound stability analysis was performed for the breakwater using the revised average annual wave climate. The stability performance of the breakwater over its project life was found to exhibit greater than expected damages, beyond even those predicted for the updated wave climate.

Greater than expected damages were experienced along 41 percent of the breakwater length. An average of 14 percent of the overall breakwater template cross-sectional area has required replacement along the entire structure length. These greater-than-expected damages indicate that either another failure mechanism is involved, or damages are greater than the mean predicted by theory.

The greatest changes in the structure occurred for the time period 1975 through 1989. This time period also exhibited higher than normal water levels and severe storm events.

The highest concentration of maintenance was placed on Segments 1, 4, 2, and 3 in order of magnitude. While greater amounts of maintenance were placed per foot in Segments 1 through 4, those segments also display areas greater than template, indicating more stone was placed there than was needed to address damages. It has been documented that some of the maintenance stone was placed along the breakwater in an attempt to reduce transmission in the harbor.

The damage magnitude takes this factor into account by subtracting out the excess area. When that is done, Segment 1 remains the most critical in spite of its positive area gain, and Segments 5, 7, and 8 show high rates of damage.

Original design and present design theory damage estimates underpredicted wave damages. Breakwater damages due to waves sustained by the harborside of the breakwater were much greater than estimated in the original design due to the wave climate and the water level being more severe than predicted.

The high degree of variability in stability results for regular-shaped, cut stone, combined with the 1V:1.5H structure sideslopes at the Burns Harbor breakwater, may have contributed to wave damages being greater than originally expected.

The extreme randomness of the stability performance of a rubble-mound structure may also be responsible for some of the higher damages. With regard to Segment 1, it is expected that stability conditions on the head of a breakwater will be more severe than on the trunk of the structure due to diffracted waves and additional overtopping.

The settlement analysis indicated normal potential settlement predictions. While settlement may be accountable for a portion of the missing stone, determining what maintenance, if any, may have been placed to address settlements on the order of 1 to 2 ft (0.3 to 0.6 m) is another issue. Those magnitudes represent approximately 1/40 of the structure height and 1/6 of an

armor unit dimension. It is questionable that such gradual magnitudes would have been perceived as damages requiring maintenance.

Other settlement issues related to construction scheduling for Segments 7 and 8 on the west arm and toe scour on the harborside of Segment 1 may have contributed to greater than expected settlement damages.

With regard to future damages, predicted average damages of 2,330 tons (2,110 mt) per year (using the present wave damage design theory) appears to be an underestimate. Average annual maintenance performed over the life of the structure results in 6,010 tons (5,450 mt) per year. Future damages due to settlement should be minimal. Damages occurring in the future will most probably be very sensitive to water level.

Segments 5, 7, and 8 are currently in need of repair while Segments 1, 2, 3, 4, and 6 are greater than the design template in area. Although maintenance is not required at present, Segment 1 will require special attention in future years due to its vulnerable location and performance history.

Cross-sectional analysis performed for this study was restricted by several potential errors. Use of several sets of survey data obtained by different methods over a period of time introduces potential analysis error. To allow a statistical analysis of breakwater segments, a lower limit of -30 ft (-9.1 m) LWD was established for the cross section. This prohibited analysis of the variable toe of the breakwater.

Recommendations

The MCCP investigation for Burns Harbor breakwater involved numerous aspects of the breakwater's design and maintenance. Results of this study include recommendations for survey techniques for rubble-mound structures to allow for future analysis and specific recommendations to improve the stability of Burns Harbor breakwater.

Historical investigation into the performance of the breakwater was complicated significantly by inconsistencies in the survey documentation. Changes in breakwater stationing and survey monument designation made cross-sectional area comparisons problematic. In particular, alternate use of a center line and a baseline for different sets of survey data made conclusive delineation between the harborside and the lakeside of the structure impossible. Future surveys of rubble-mound structures should be carefully documented over time to provide a consistent measurement of performance.

A comparison of survey methodology through the course of the study has shown that the method used for the underwater portion of the breakwater may produce a distorted underwater representation. Two types of underwater surveys were used on the Burns Harbor breakwater: fathometer and sounding basket.

Recent experience with the sounding of steep rubble-mound slopes using a fathometer has shown acoustic soundings to be susceptible to spatial distortion. The fathometer survey technique on one survey in particular produced a distorted, or inflated, below-water cross section which in effect indicated a cross section in better condition than was actually the case. Mechanical means of survey such as the metal sounding basket were more consistent in their results. If fathometer surveys are used, special care should be taken to utilize the appropriate equipment and proper tuning methods.

Based on the results of this study, modifications to the Burns Harbor breakwater cross section would improve long-term stability of the structure. Current rubble-mound stability guidance recommends the use of a design wave between the significant wave, $H_{1/3}$ and the average of the highest 10 percent of the waves, $H_{1/10}$. It is recommended that $H_{1/5}$ be used for Burns Harbor breakwater to establish a more conservative design without making construction costs excessive.

Establishing the design wave as the 10-year return interval event and using $H_{1/5}$, the design wave would be 17.7 ft (5.4 m), say 18 ft (5.5 m). Using this design wave with the existing structure characteristics of a structure sideslope of 1V:1.5H and a specific weight of stone of 145 pcf (2,323 kg/m^3), the required armor stone weight would be 23 tons (21 mt) and 30 tons (27 mt) on the trunk and head of the breakwater, respectively.

There are several possible modifications that would provide a more stable design for the front slope of the structure with a smaller armor stone. Increasing the specific weight of the stone and flattening the lakeside structure side slope would be most effective.

Changing the stone type from Bedford limestone to quartzite, a rough angular stone, would increase the specific stone weight from 145 to 165 pcf (2,323 kg/m^3 to 2,643 kg/m^3). Quartzite has a slightly lower stability coefficient of 4.0 and 2.8 for the trunk and head, respectively. Using these values for a 1V:1.5H structure slope, the required armor stone weights would be 18 tons (16 mt) for the trunk and 26 tons (23 mt) for the head.

Flattening the structure side slope on the lakeside to 1V:2H in addition to increasing the specific weight of the stone would result in armor unit sizes of 14 tons (13 mt) on the trunk and 19 tons (17 mt) on the head of the breakwater.

The design equation that governs the stability of the front of the structure is affected by the structure slope, while the equation that governs the stability on the back of the structure is affected more by crest elevation and wave height. Since the Burns Harbor breakwater has demonstrated vulnerability to backside damages, any structural alternative that either modifies the crest elevation or the incident wave height would provide an improvement. With the new design wave of 18 ft, increasing the crest elevation from +14 ft (+4.3 m) LWD to +17 ft (+5.2 m) LWD is recommended for improved backslope stability.

Finally, if scouring at the toe of the harborside of Segment 1 is a problem, placing an underwater toe berm in this location could improve long-term performance.

Based upon the cross-sectional analysis, the entire span of breakwater Segments 7 and 8 (Stations 47+00 to 58+00) should be repaired immediately. As a minimum, the design cross-sectional template should be restored in this reach. Segment 5 should be given priority after Segments 7 and 8 have been repaired. Segment 1 should be monitored closely for stability. Special care should be taken with both the stone size and the structure slope for this segment.

Finally, this study has shown that settlement, while projected to be minor, may still play a role in the long-term maintenance requirements for a rubble-mound structure. Providing for some overbuilding of a rubble-mound structure to address the potential for minor settlement adjustments should be considered. Foundation preparation for a structure of this kind can be very important.

Determination of the applicable design wave and the impact on the crest elevation design are critical elements of the design. Harborside damages can result in significant maintenance concerns and need to be estimated in the initial design.

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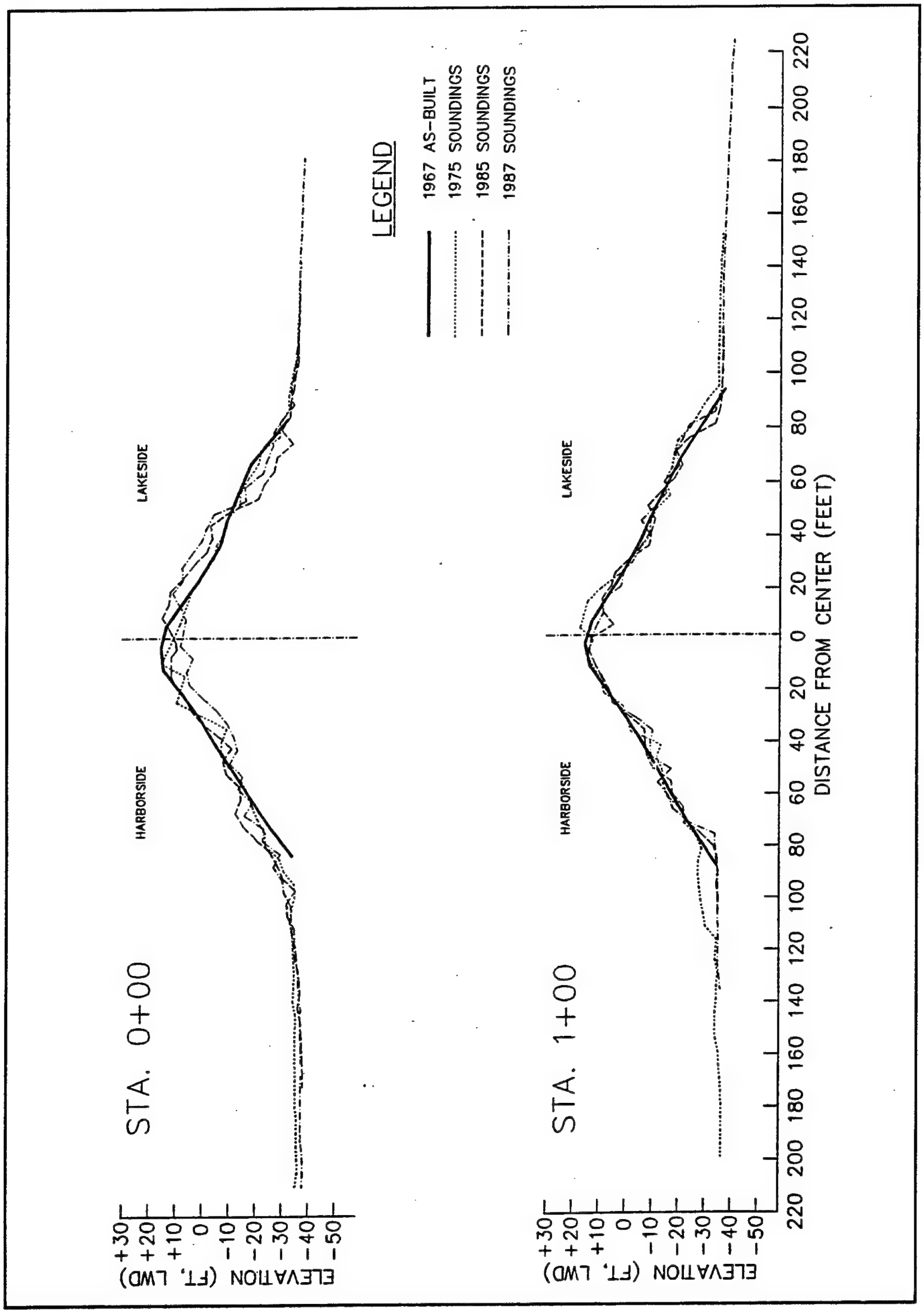
Appendix 5A
Cross-section Plots
(1967 to 1989)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1987 SOUNDINGS
ELEVATION (FT, LWD)
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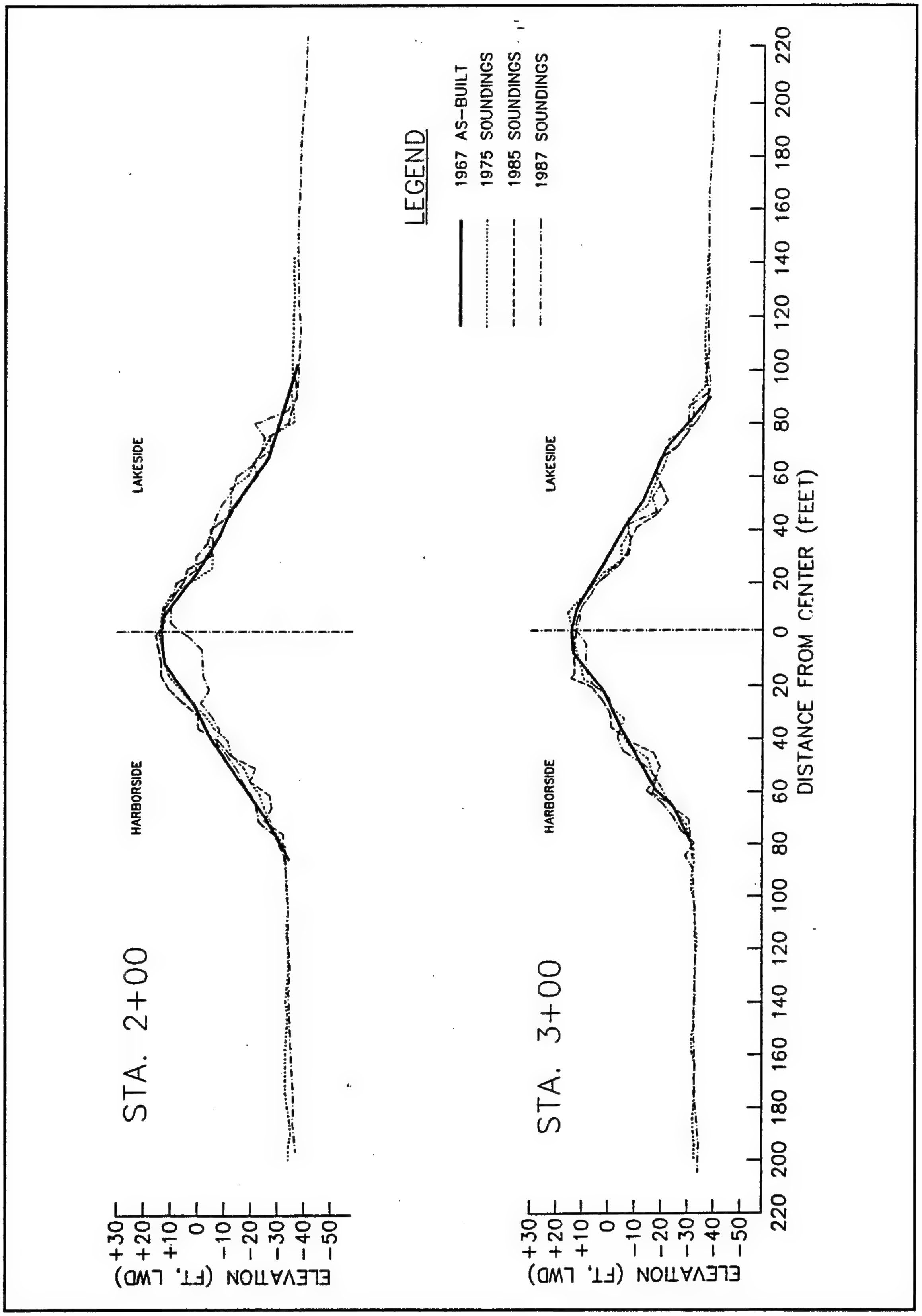
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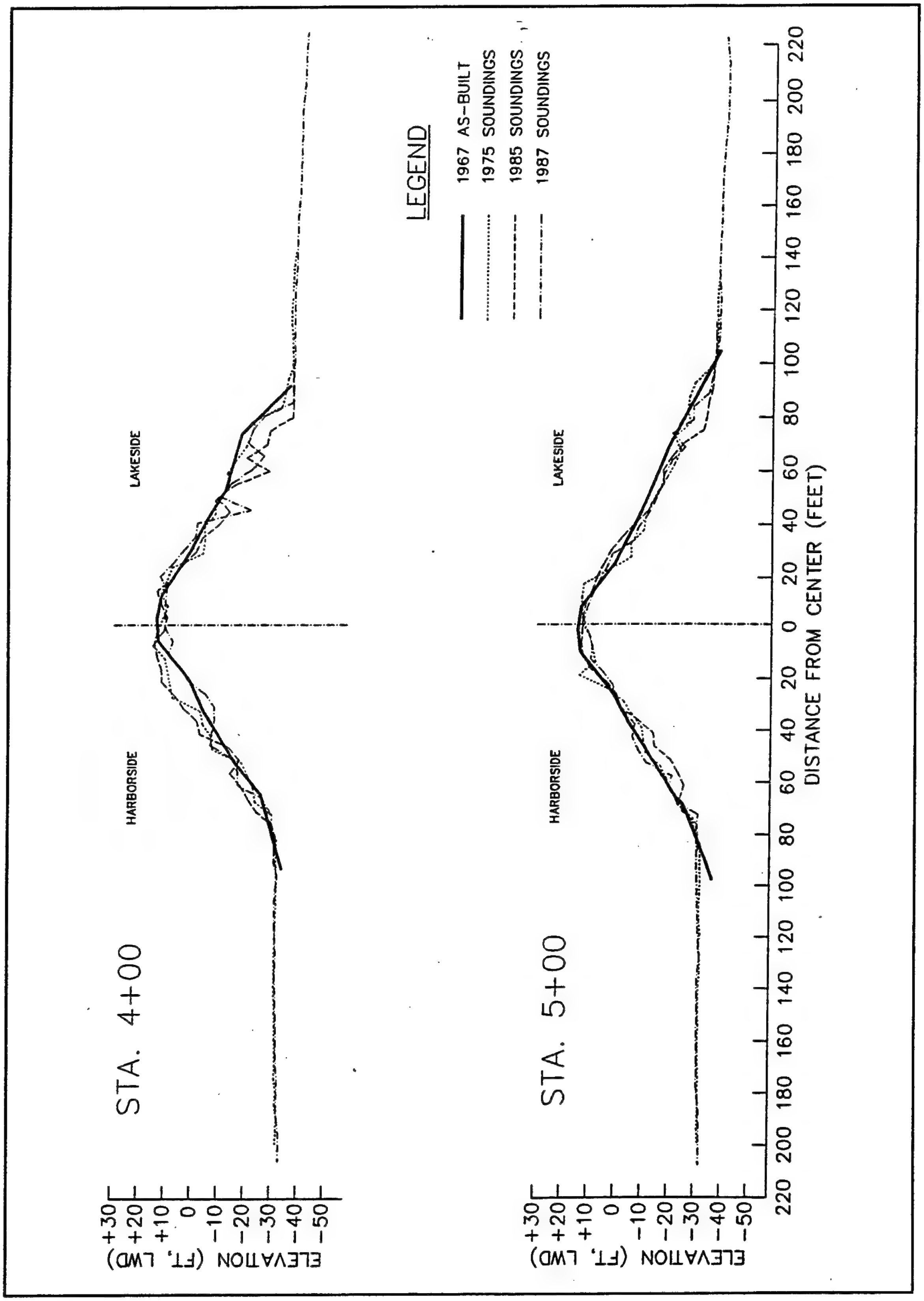

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ELEVATION (FT, LWD)
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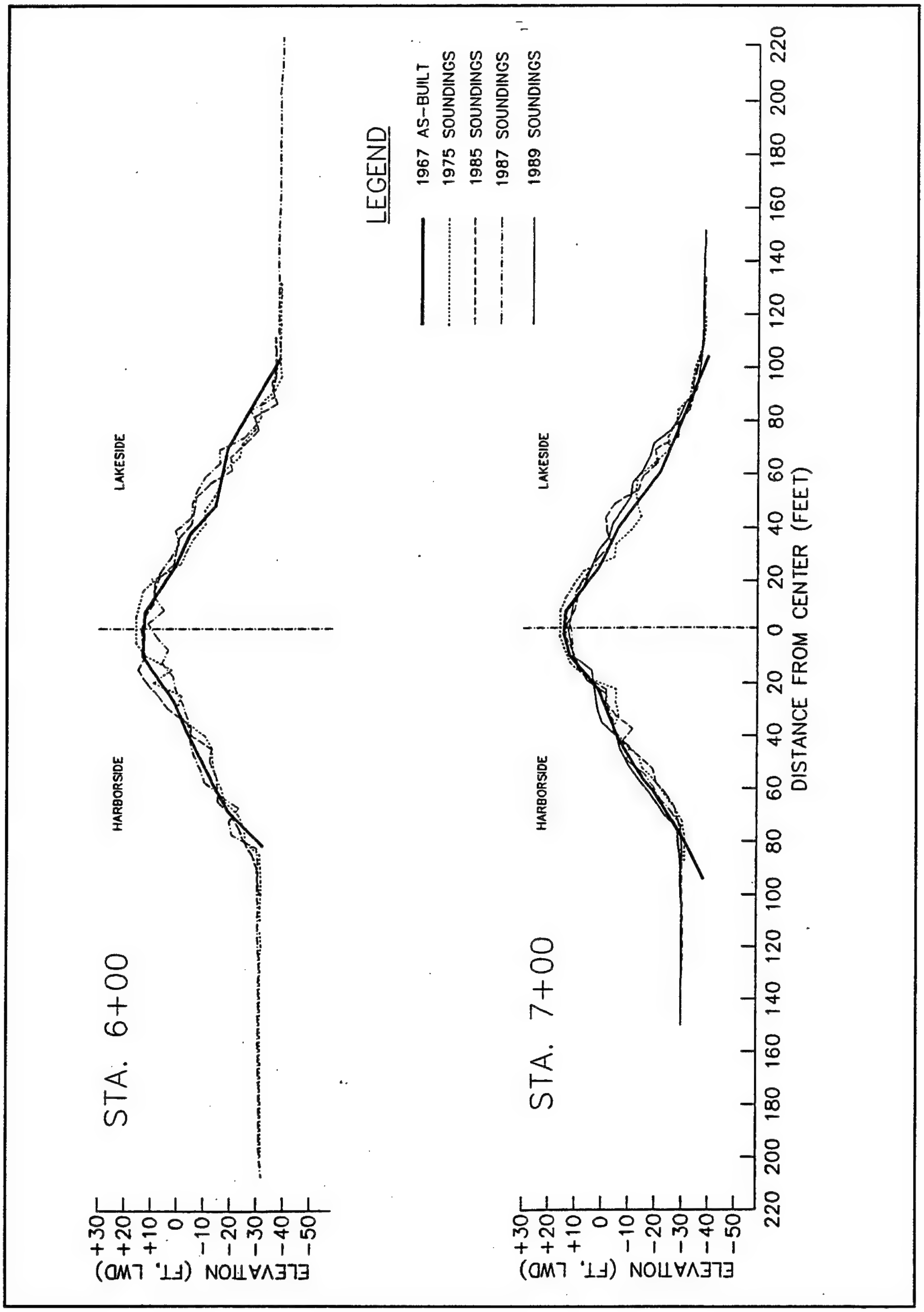
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ELEVATION (FT, LWD)
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ELEVATION (FT, LWD)
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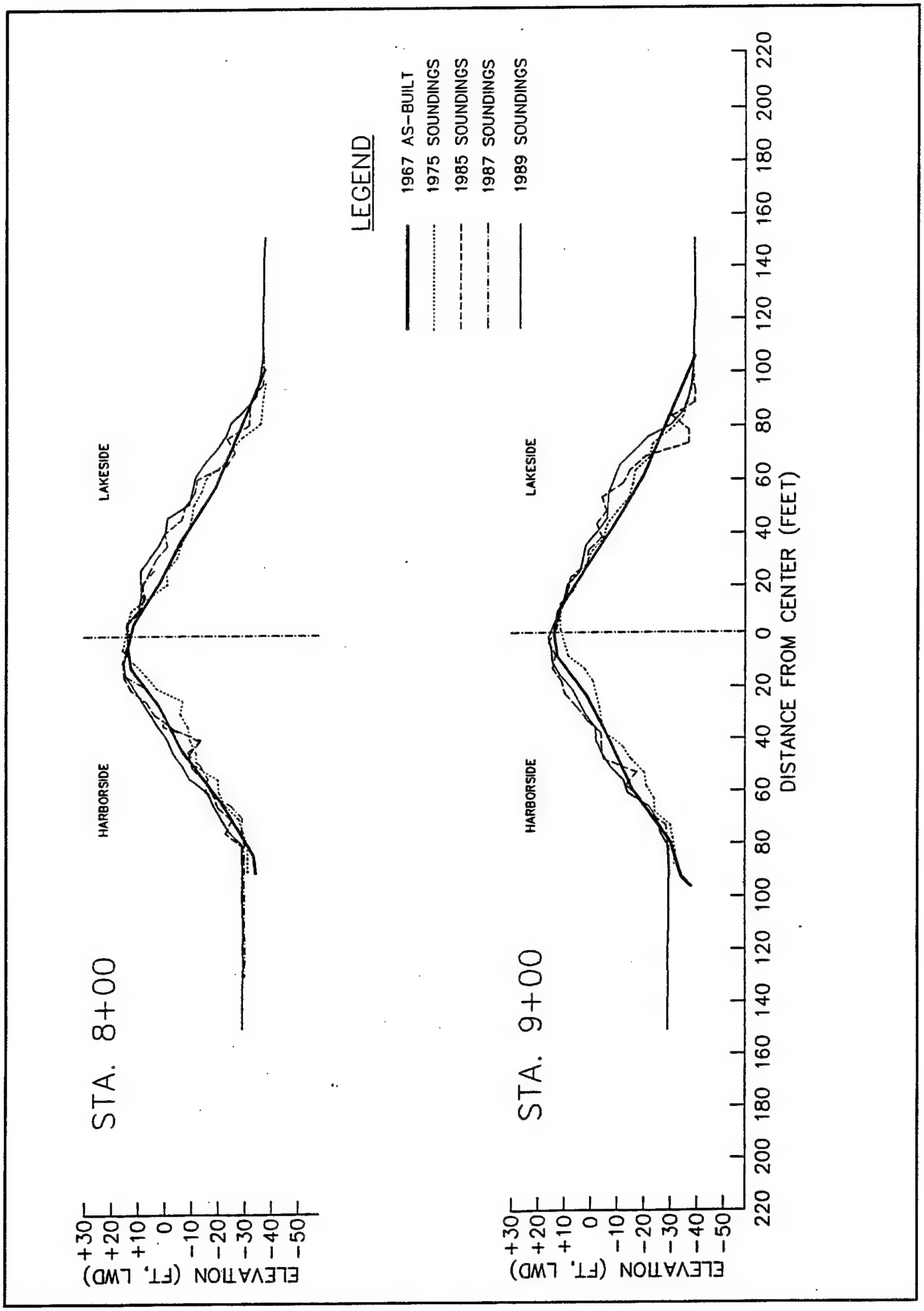
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ELEVATION (FT, LWD)
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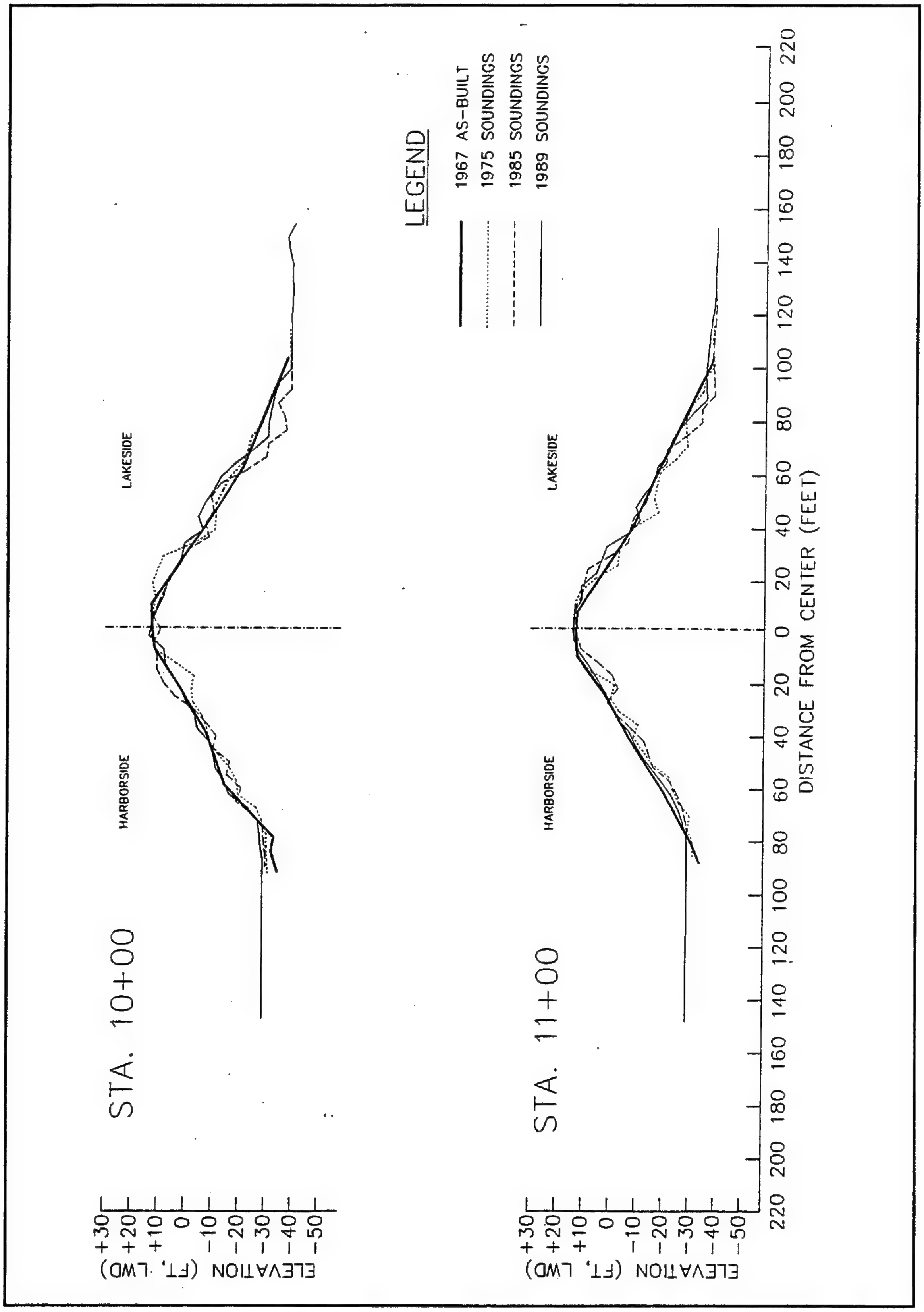
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ELEVATION (FT. LWD)
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ELEVATION (FT. LWD)
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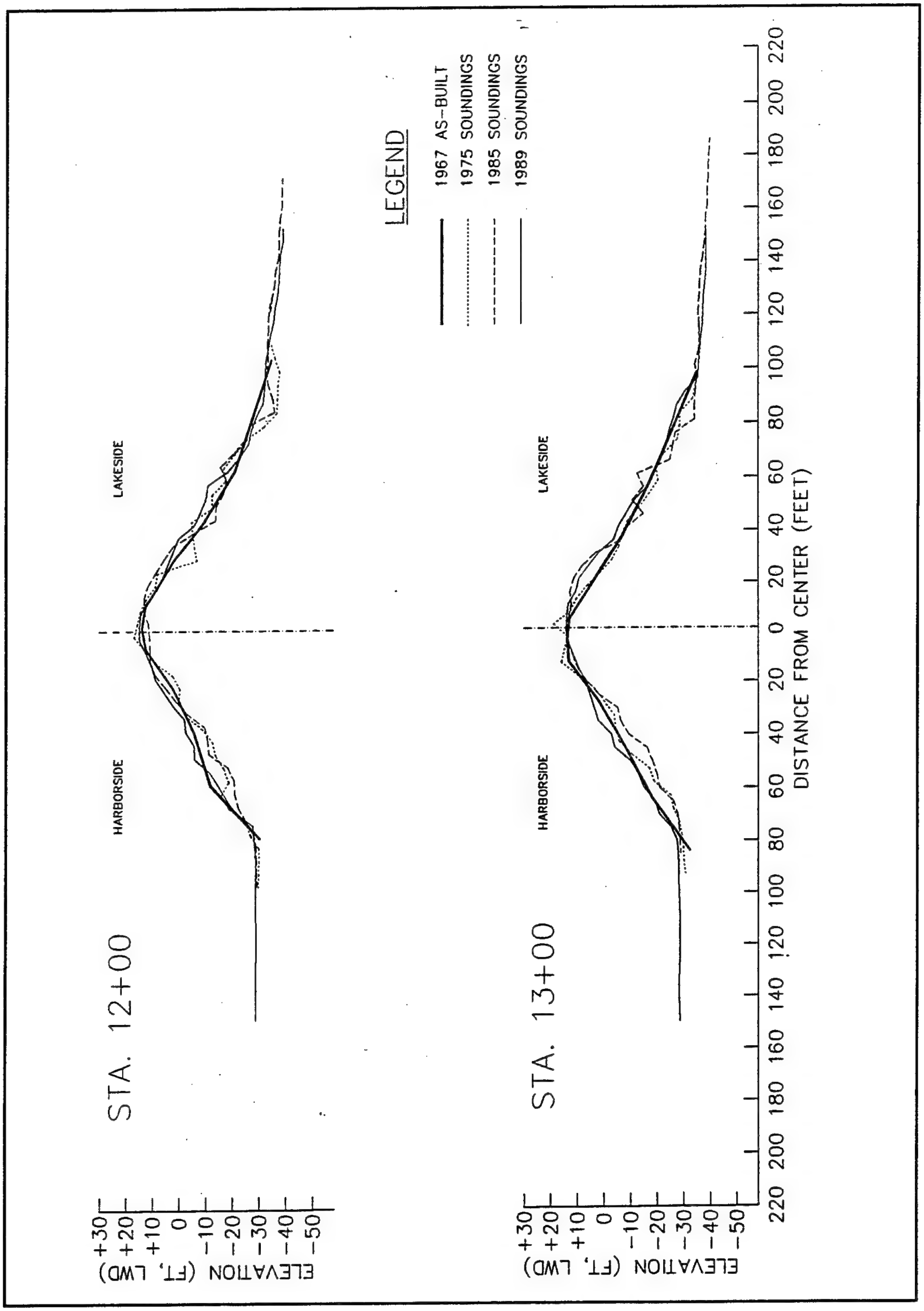
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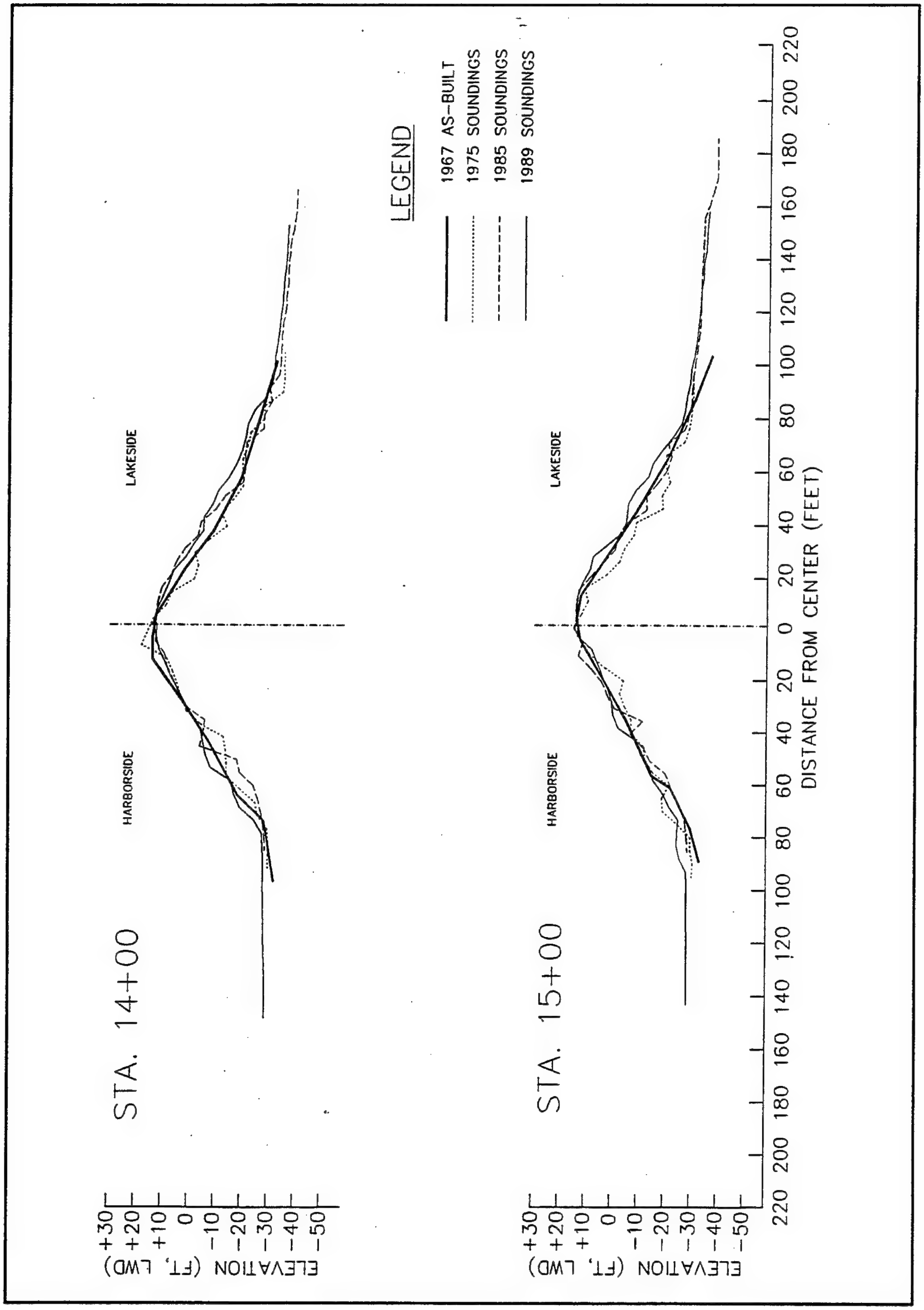
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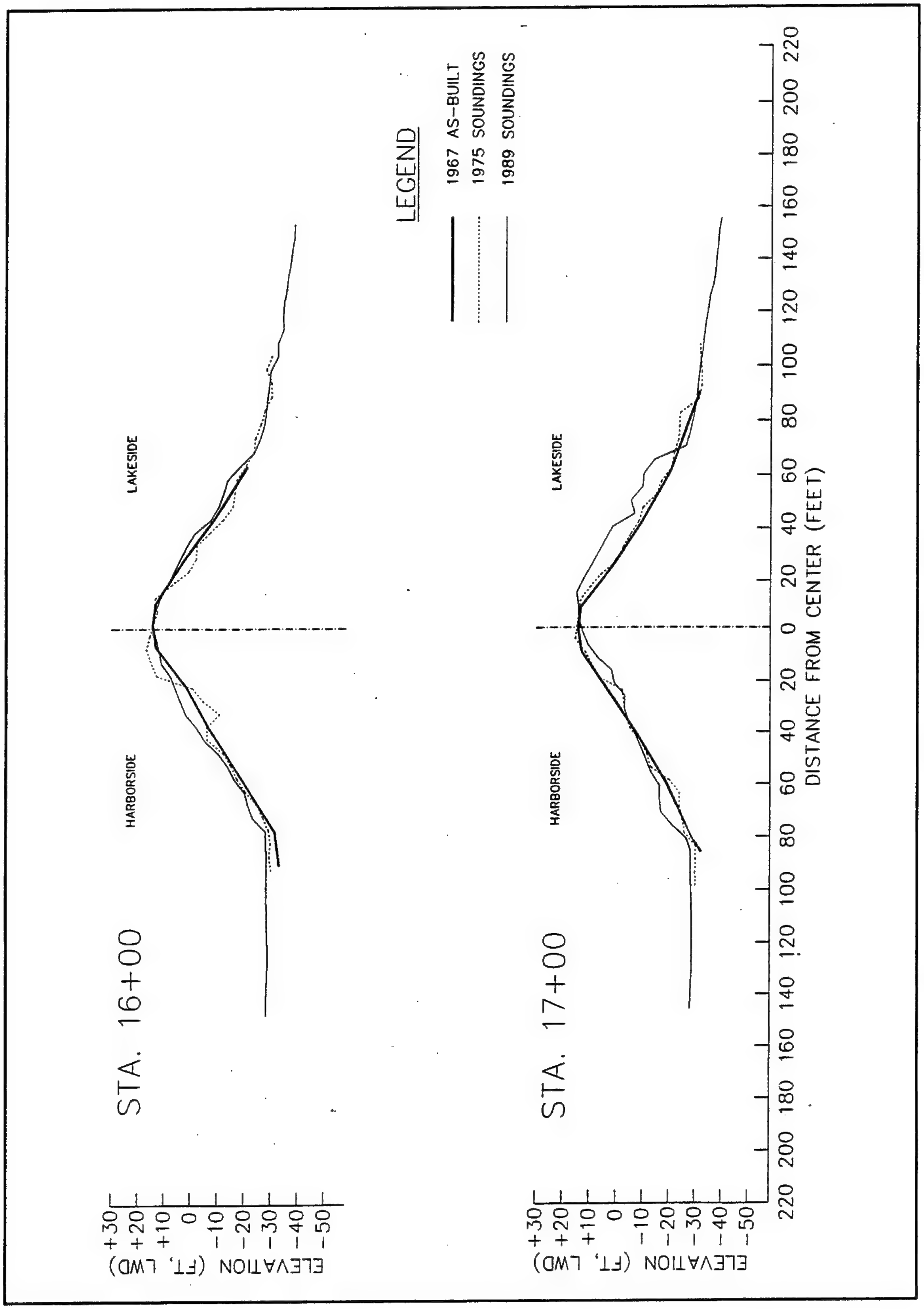
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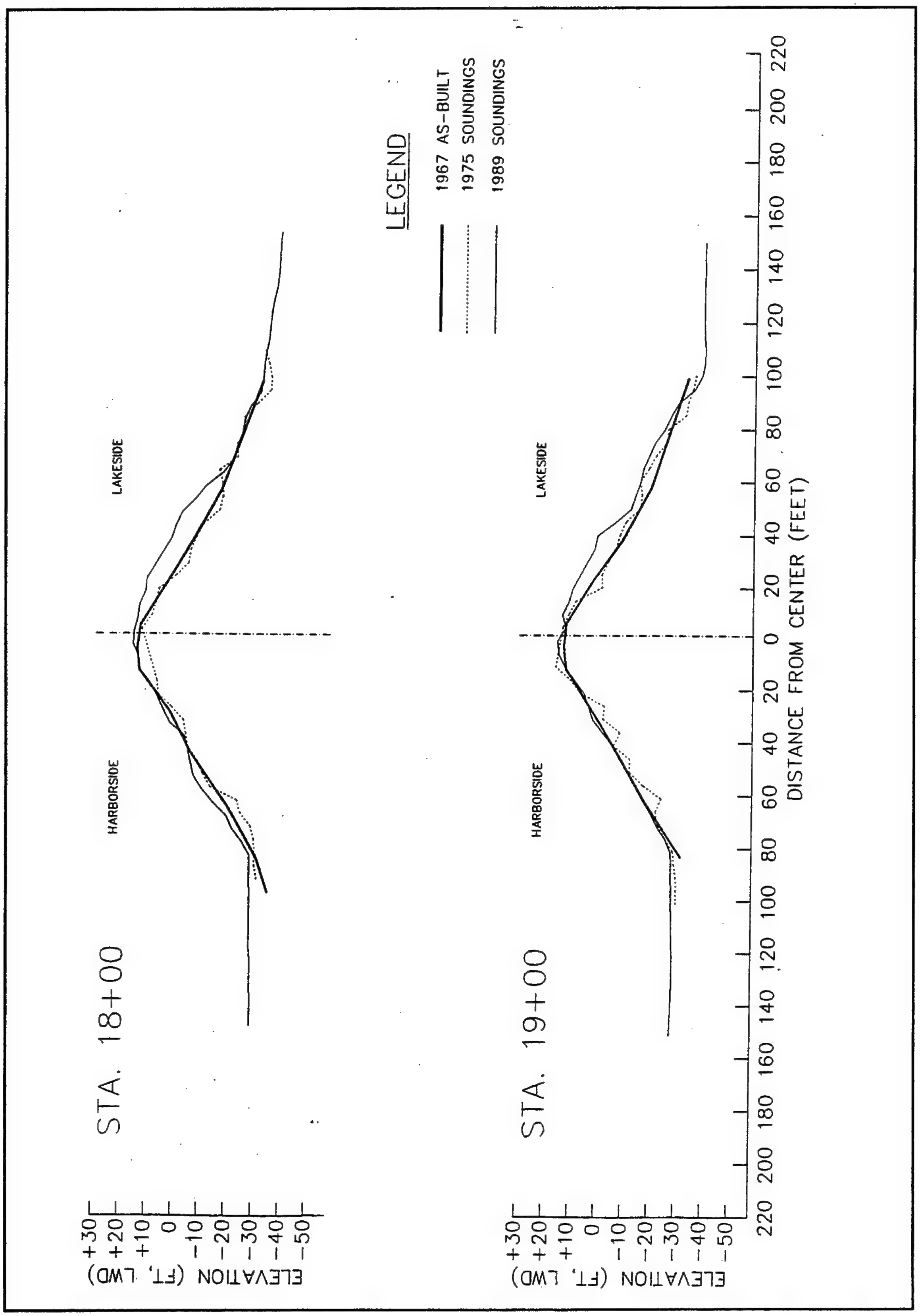

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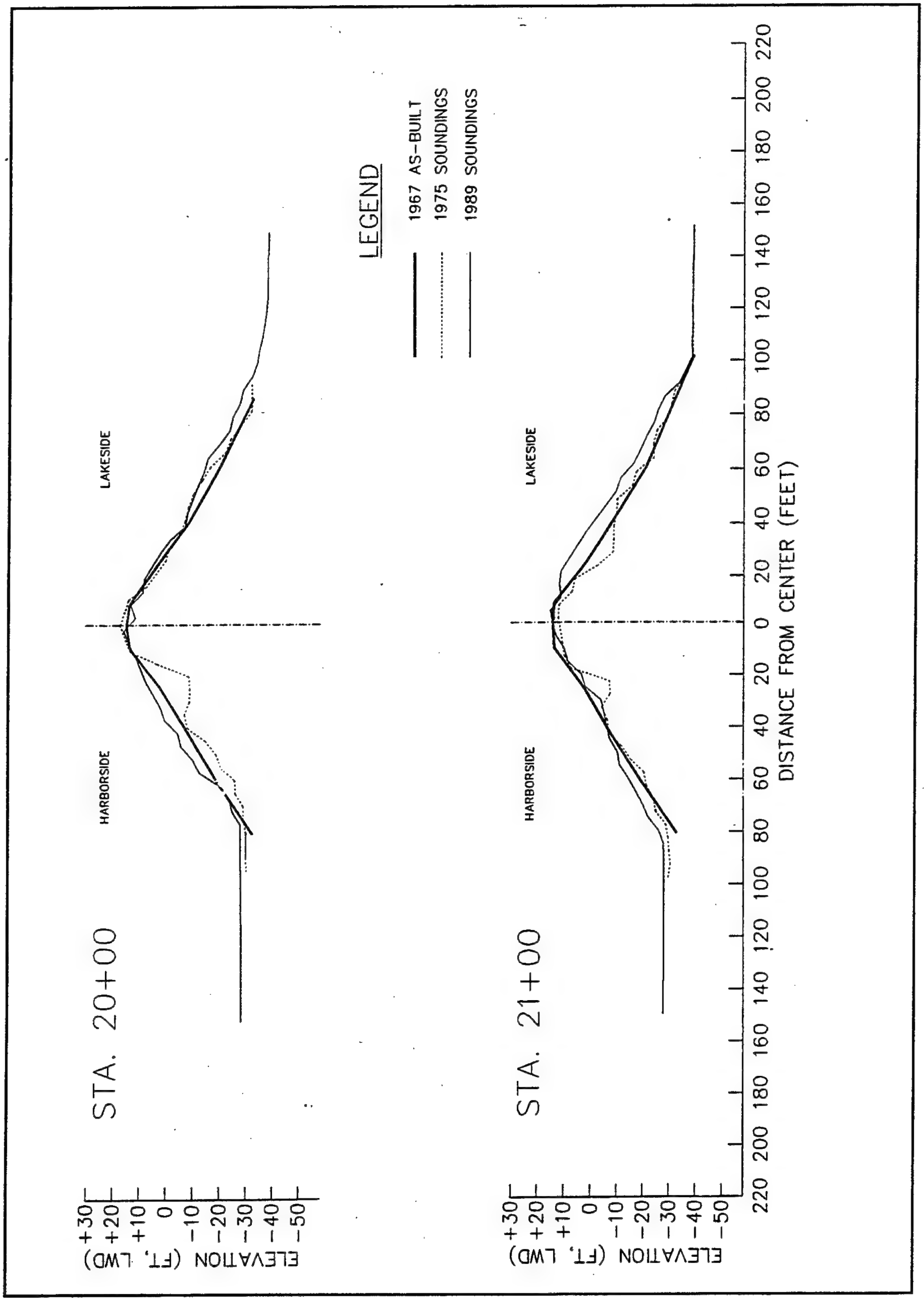
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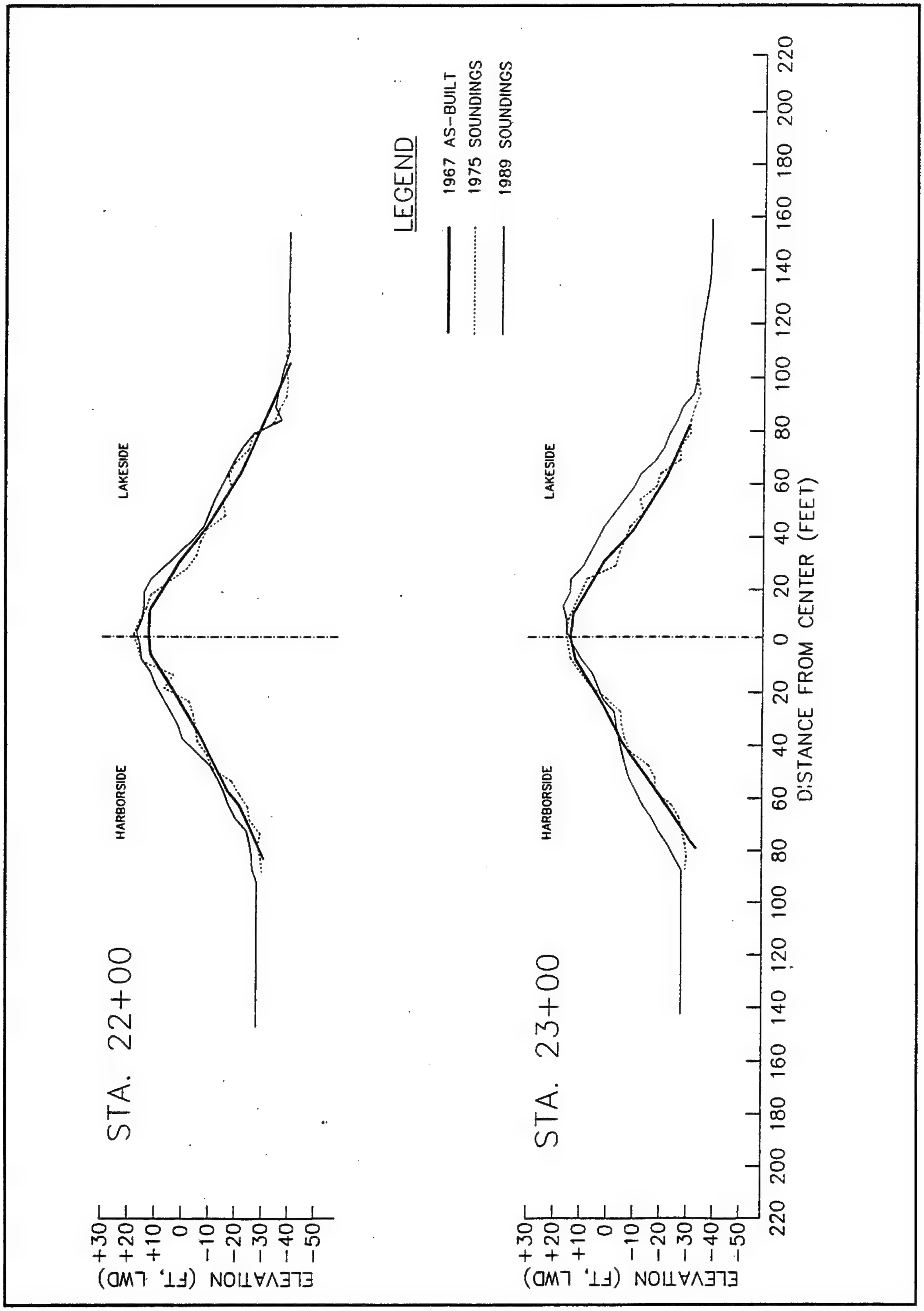
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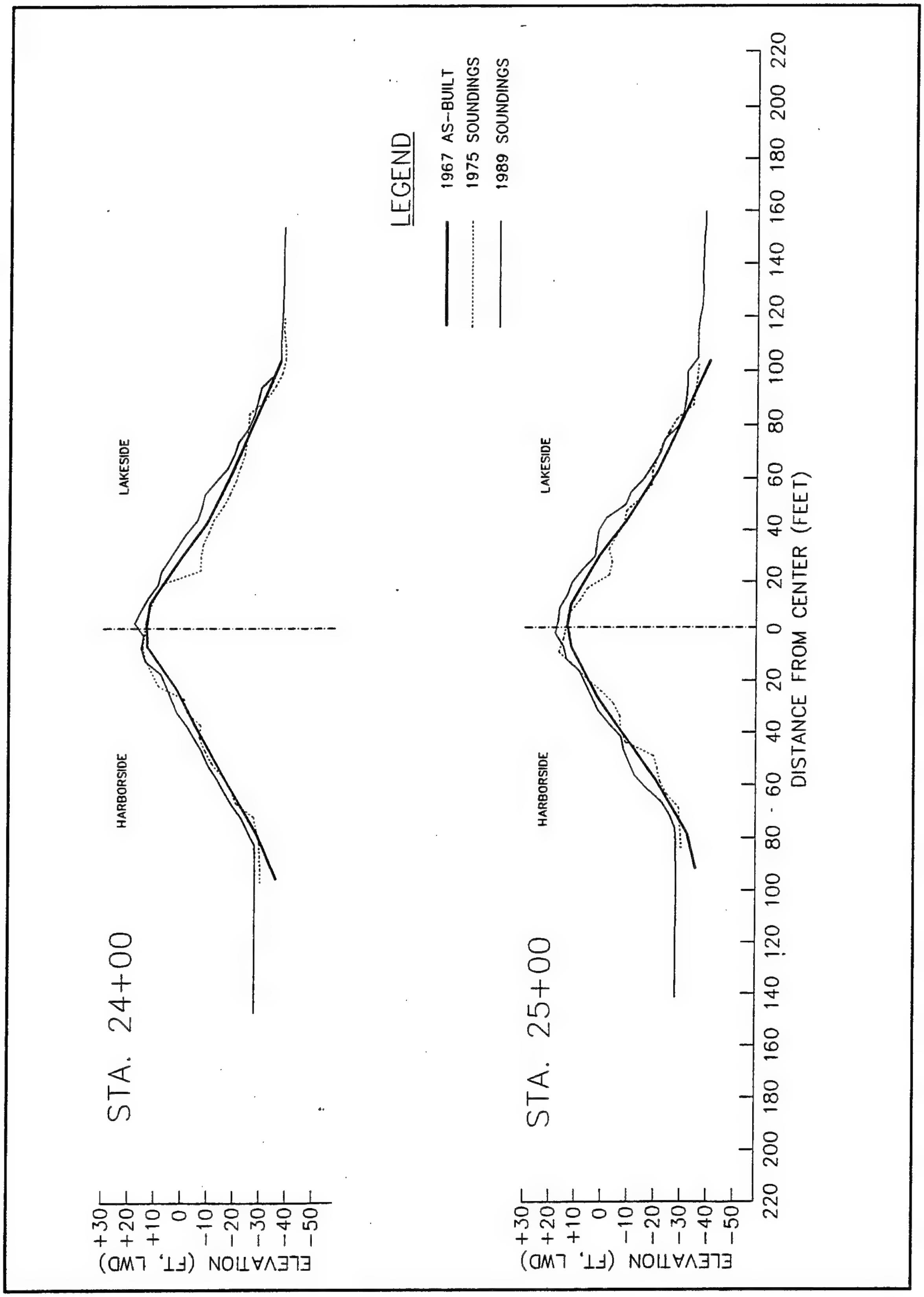
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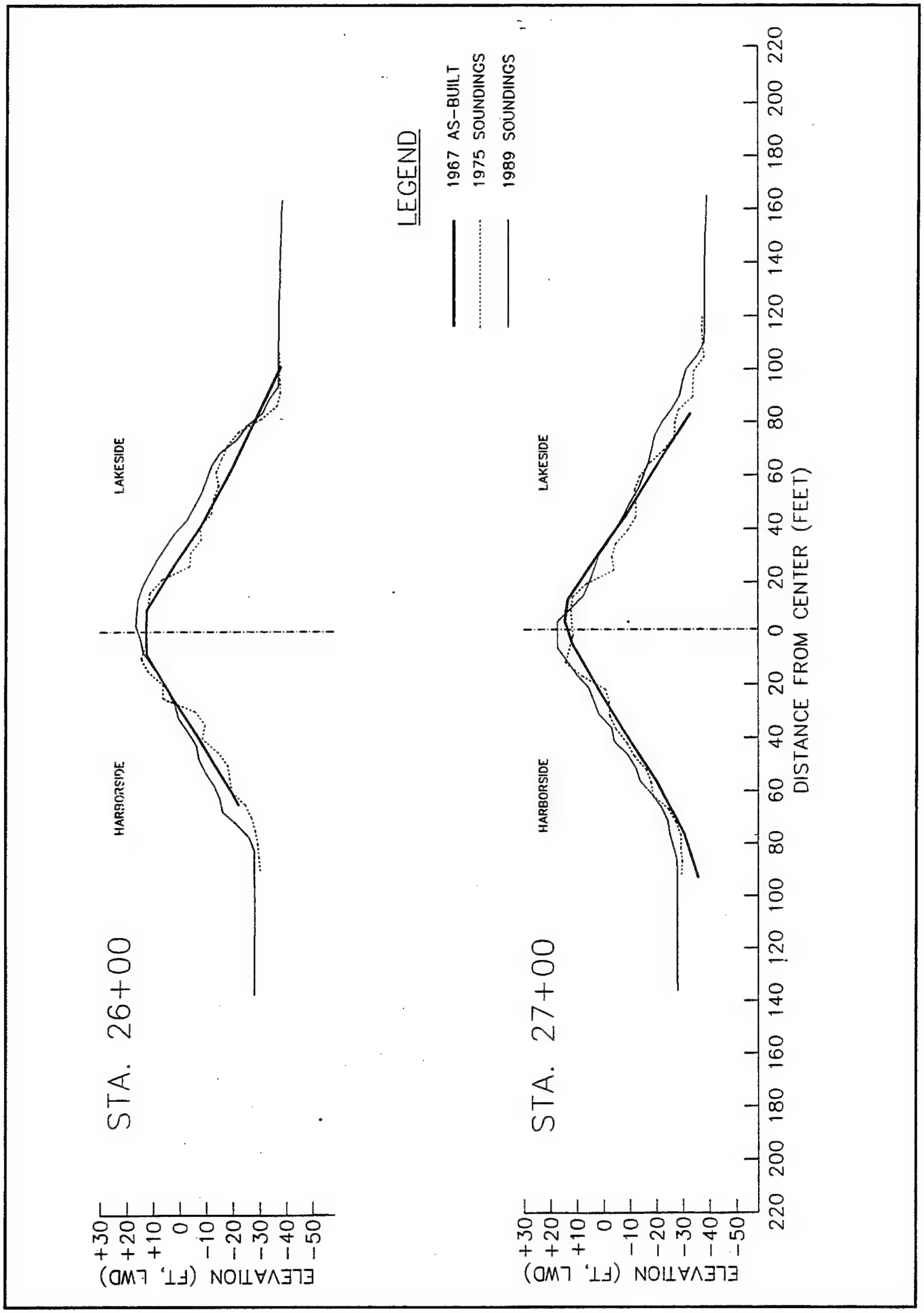

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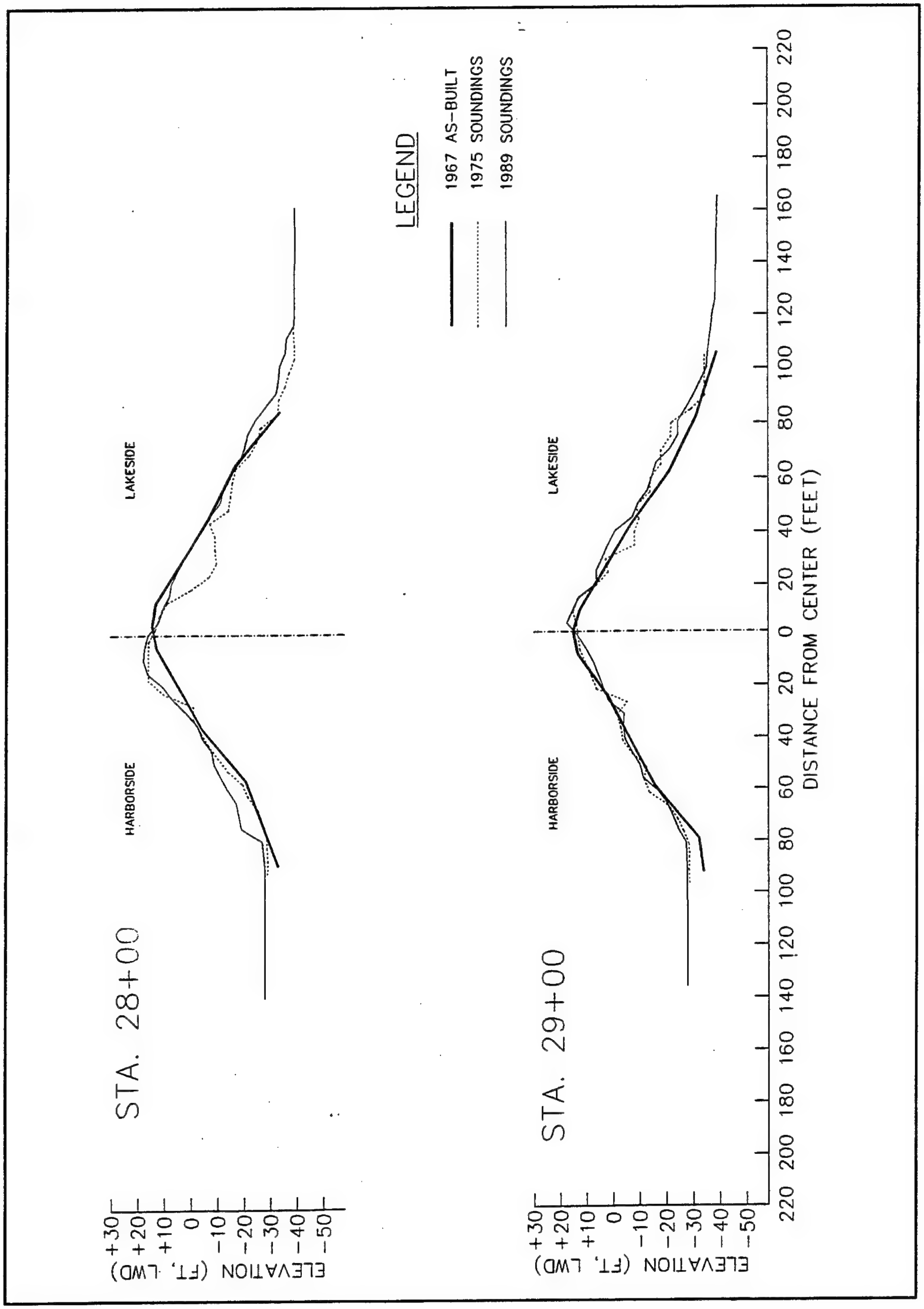

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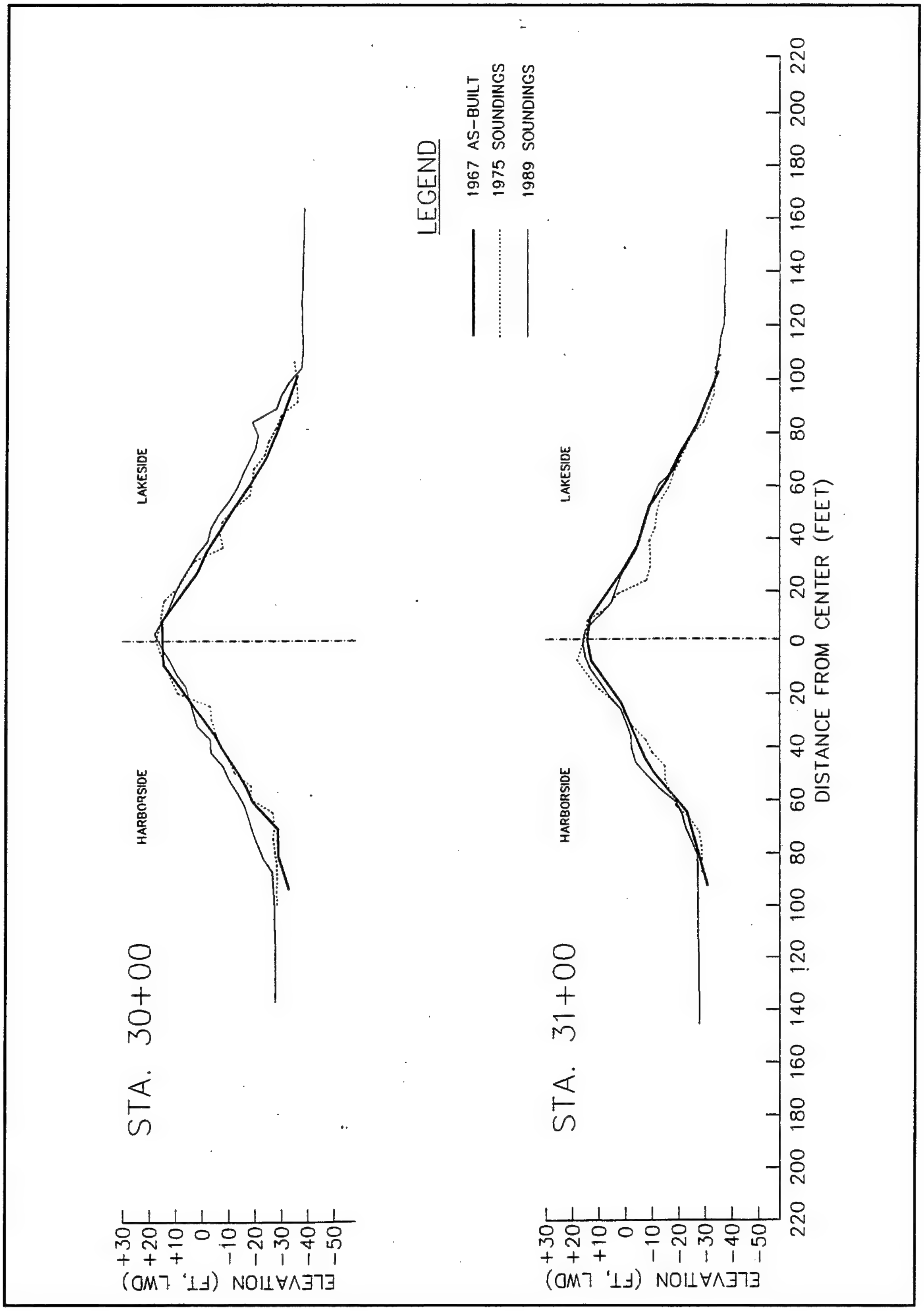
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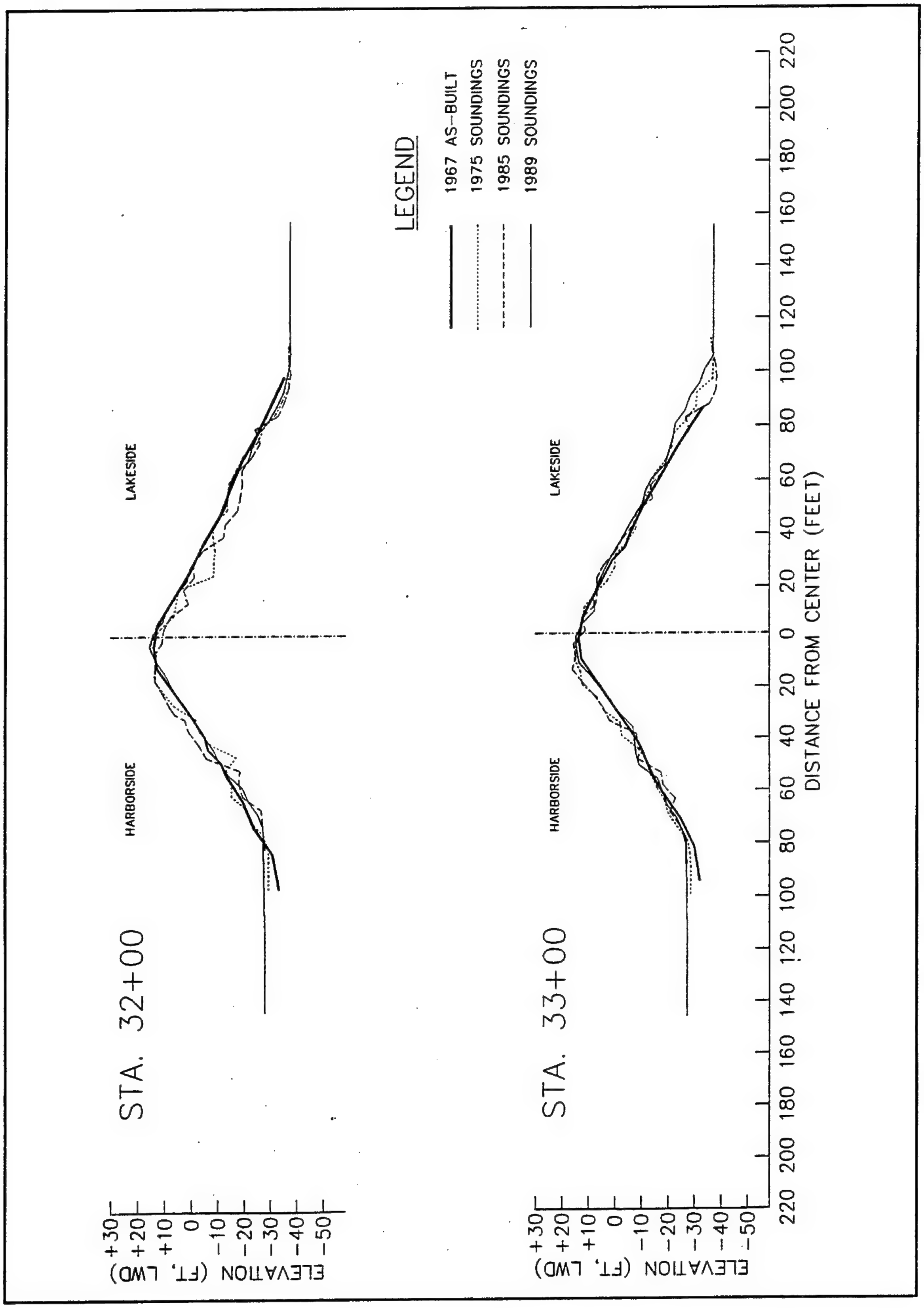
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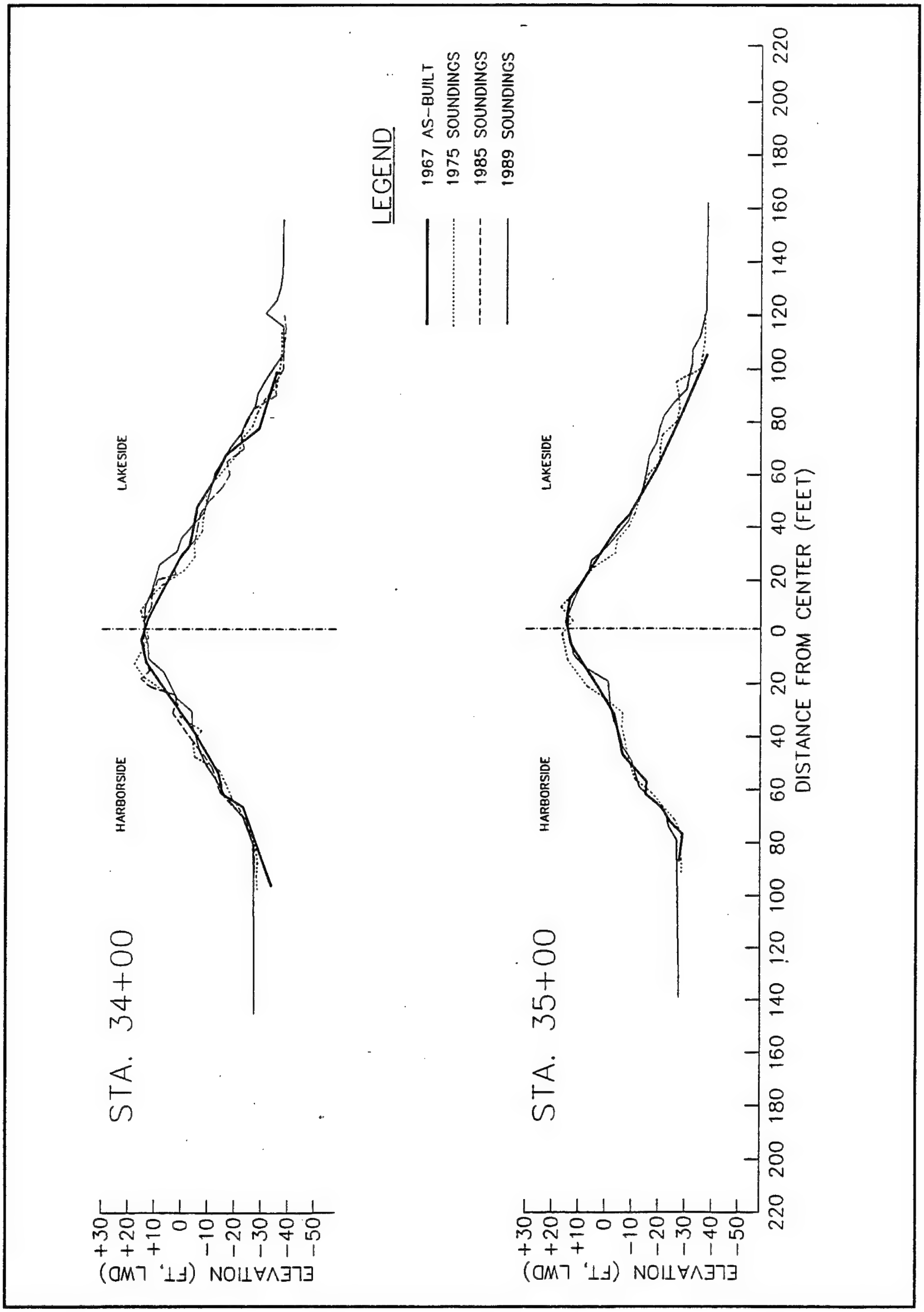
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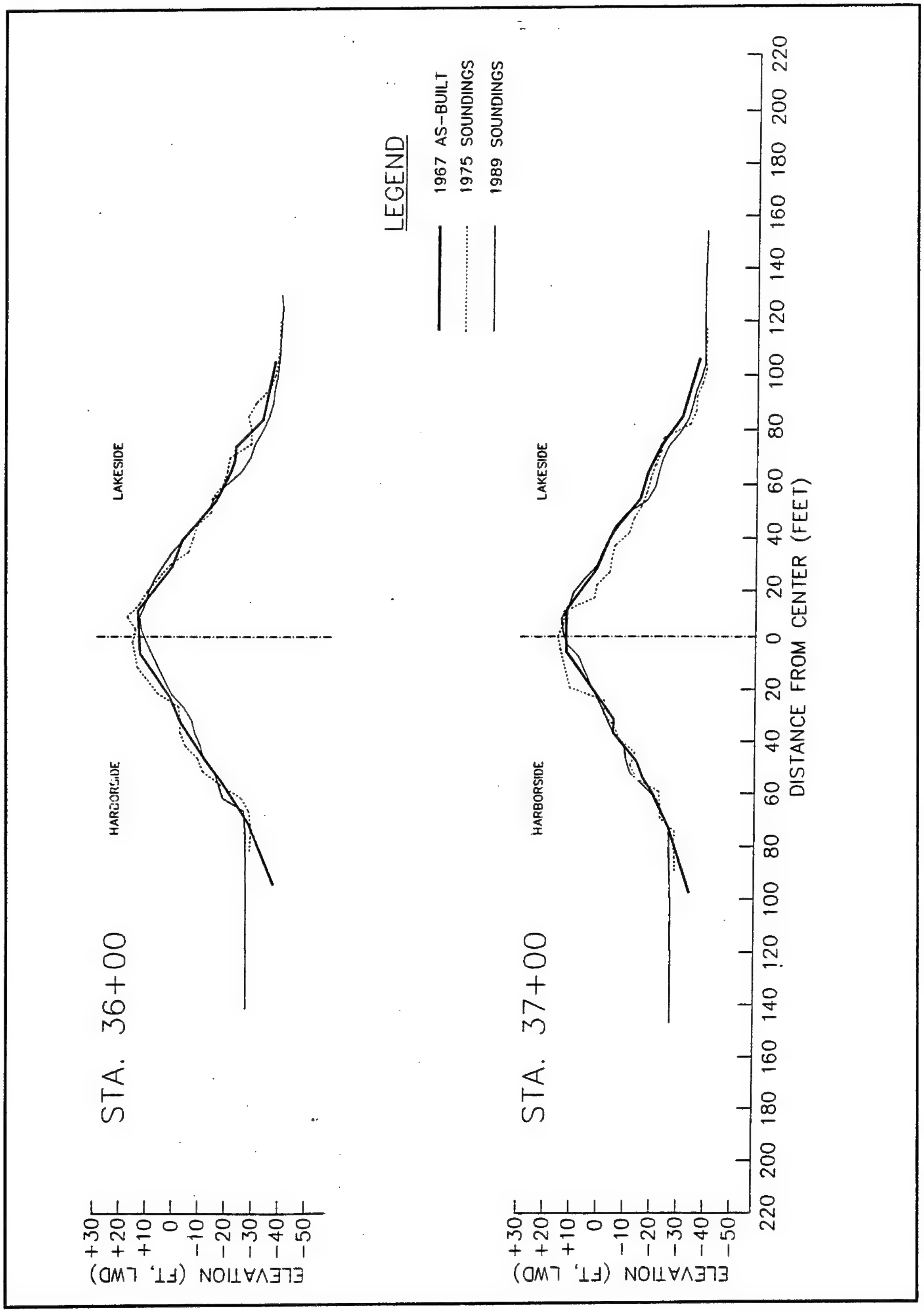

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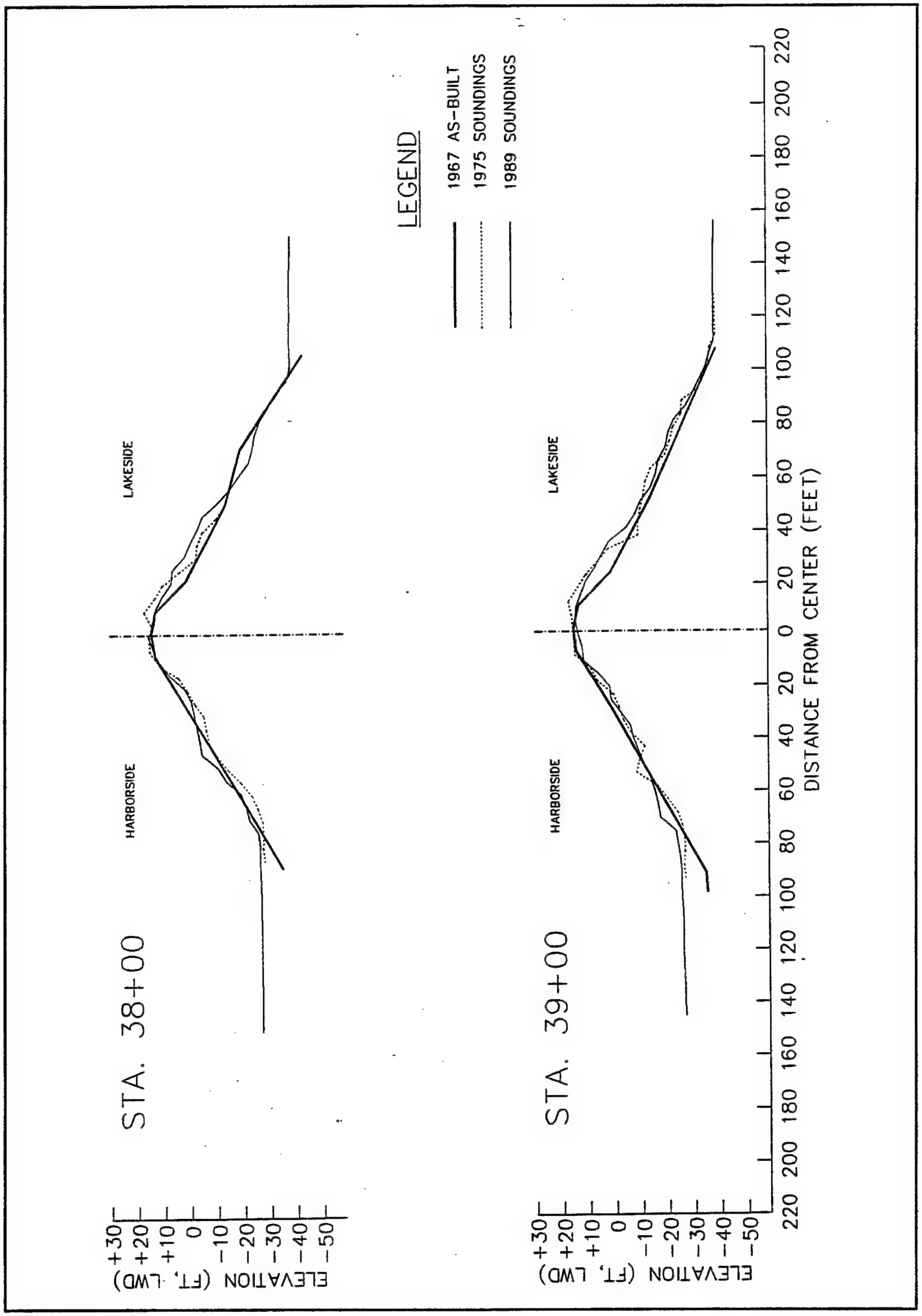

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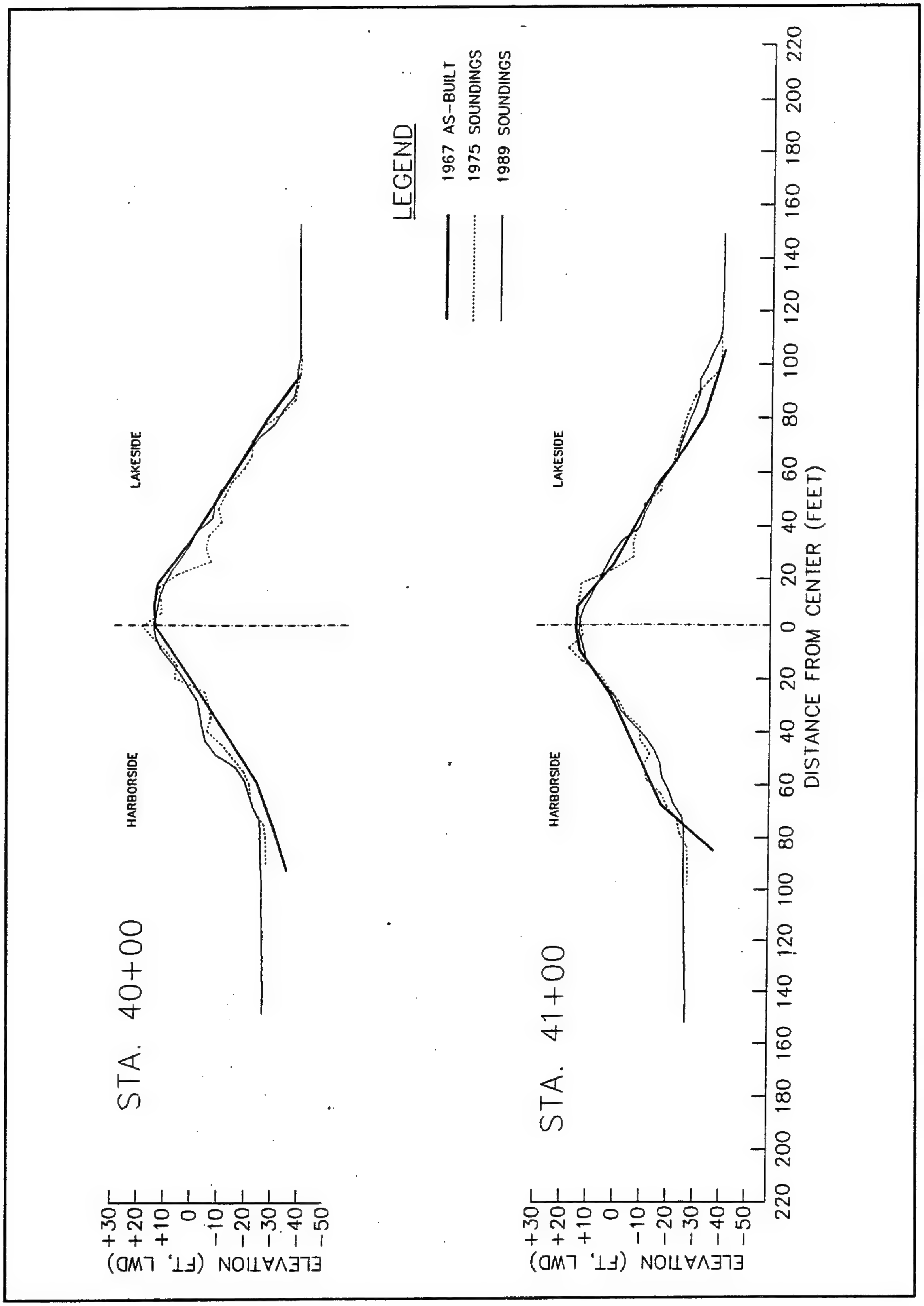
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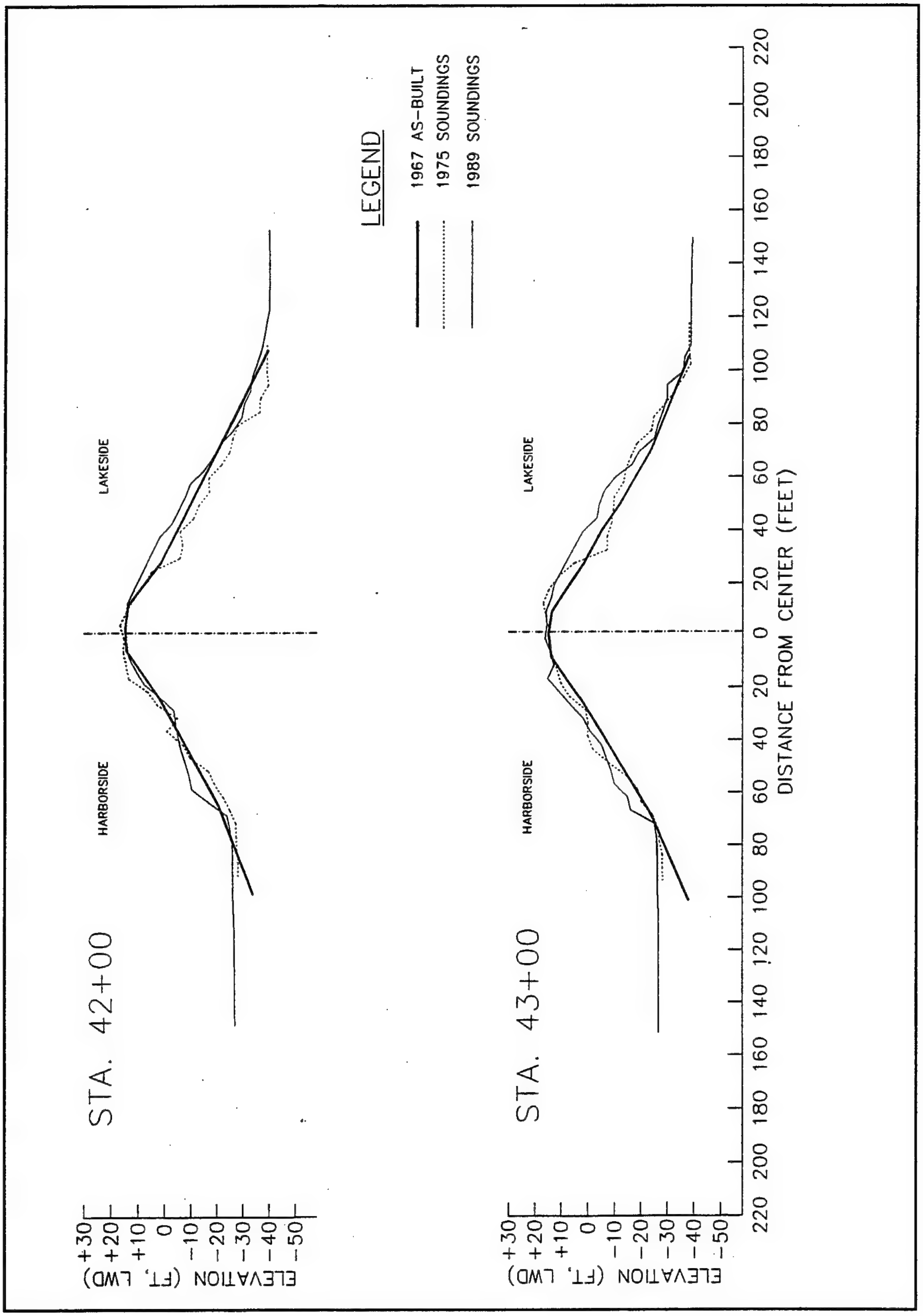
ELEVATION (FT, LWD)
DISTANCE FROM CENTER (FEET)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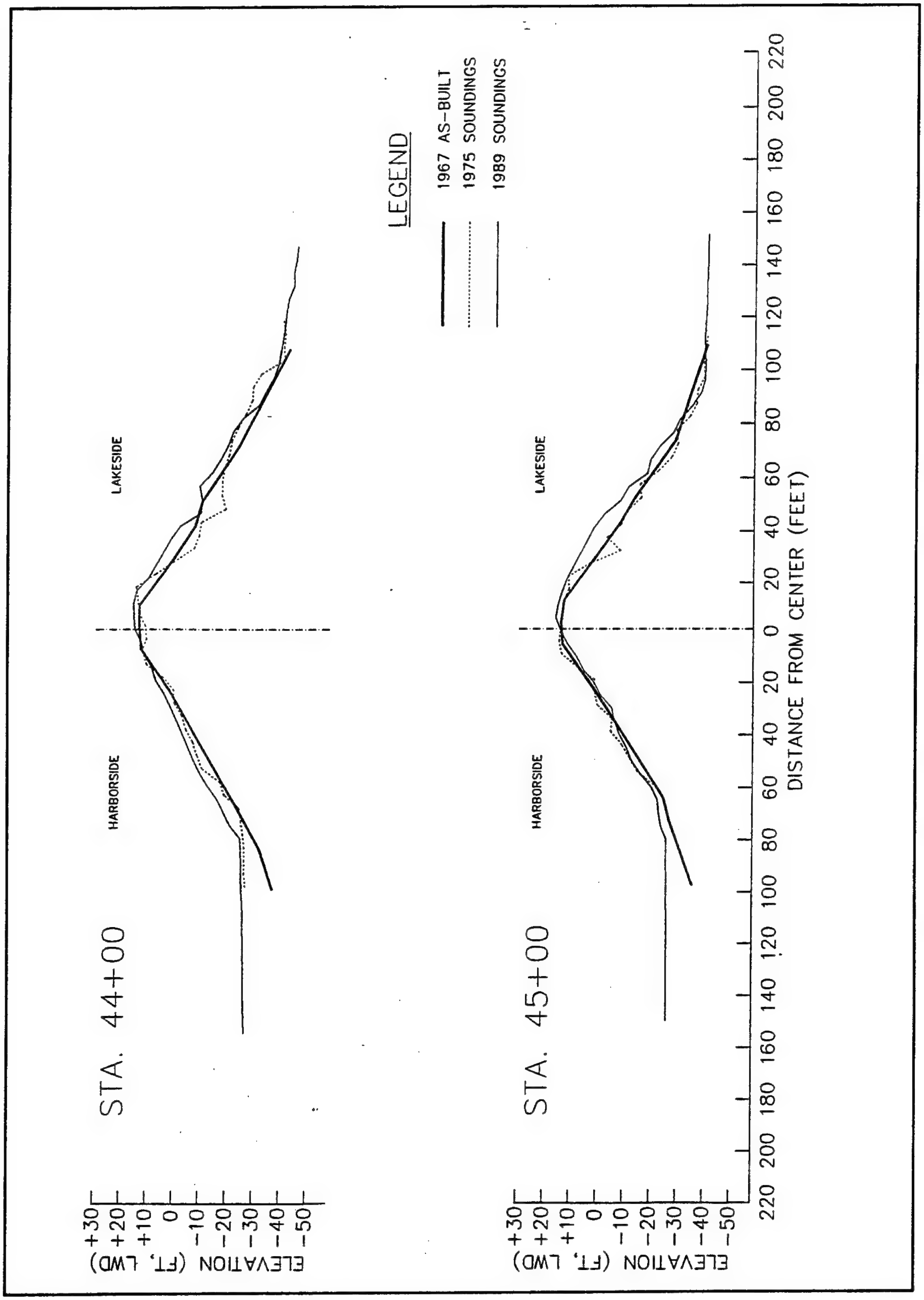

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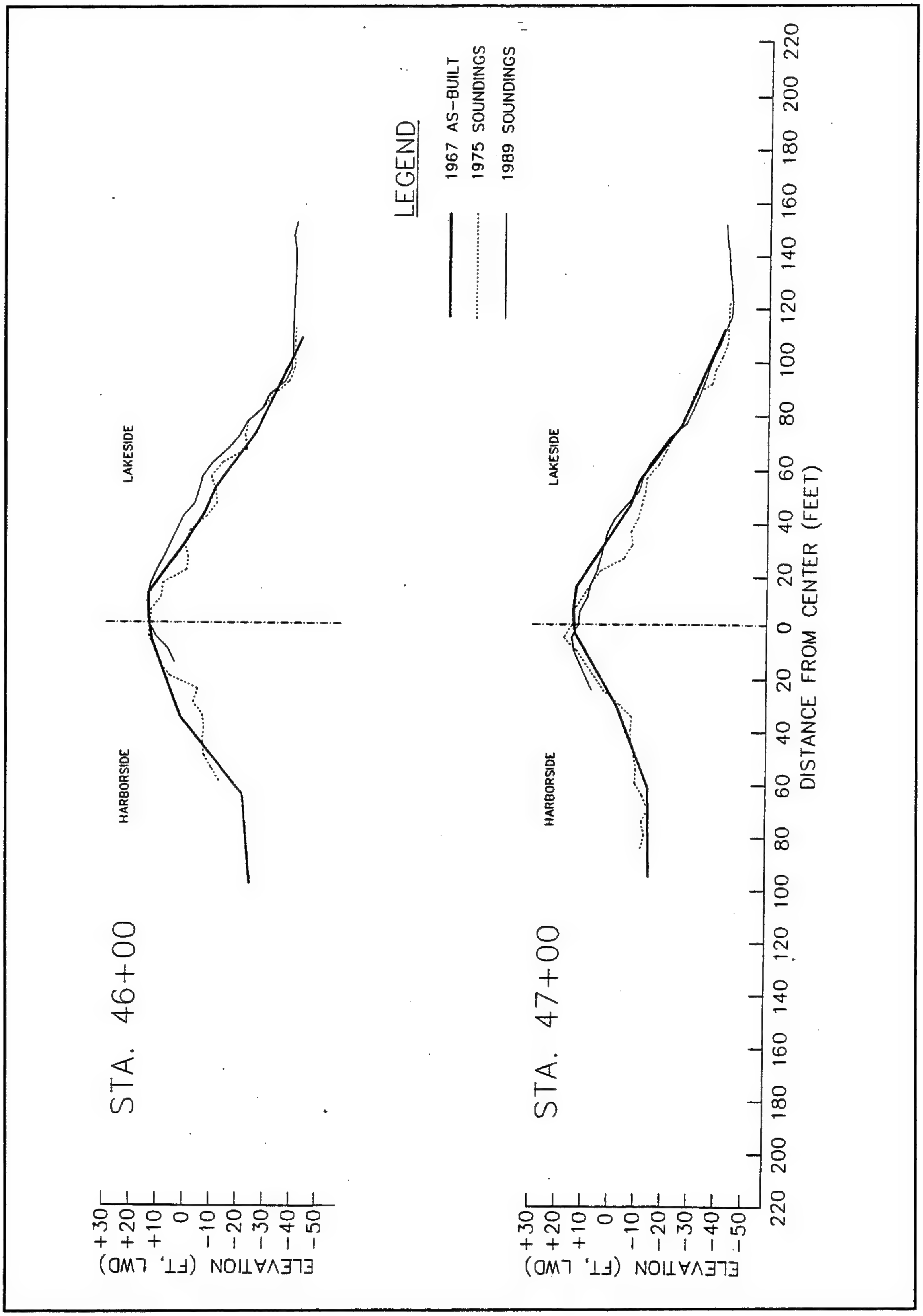
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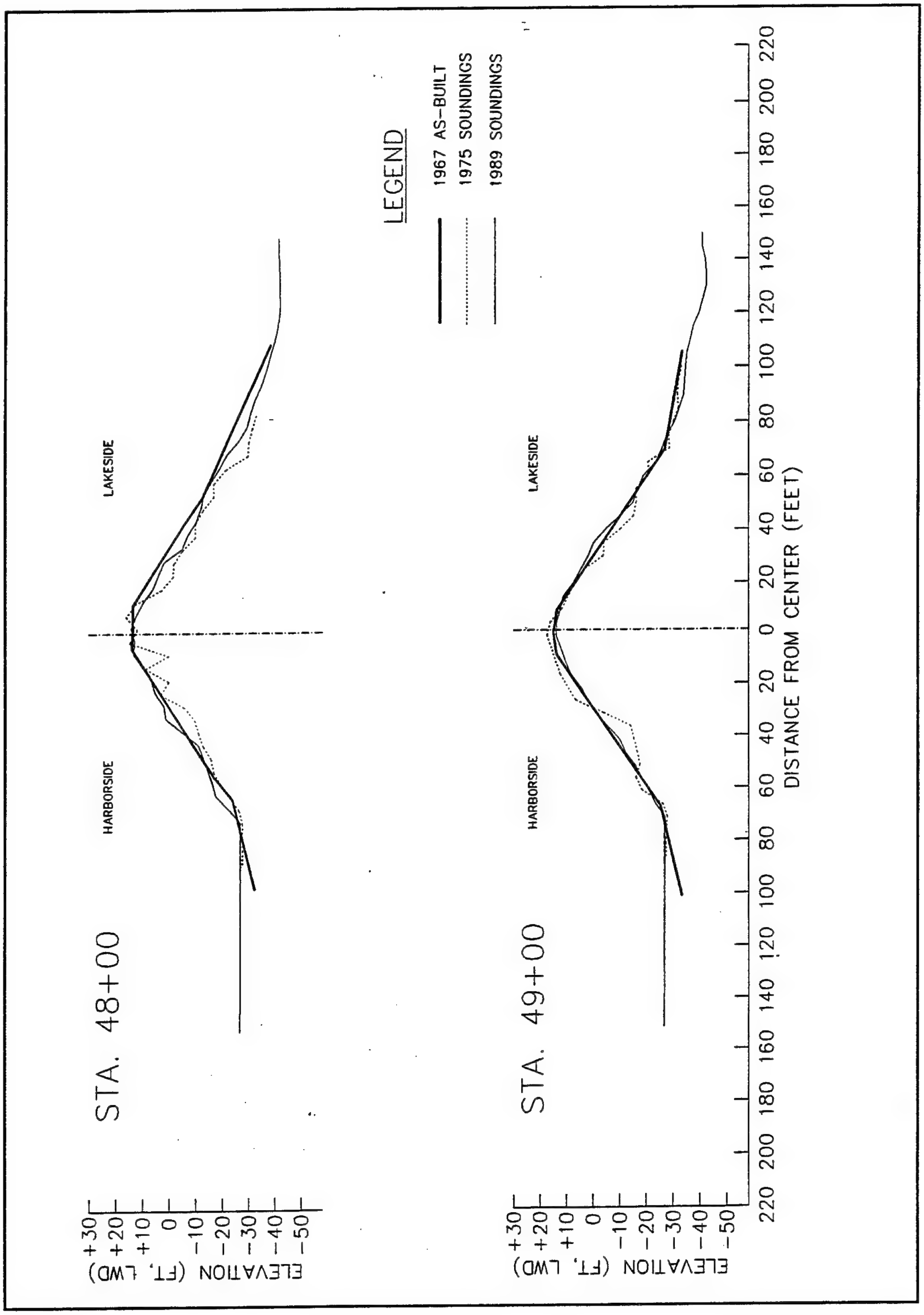
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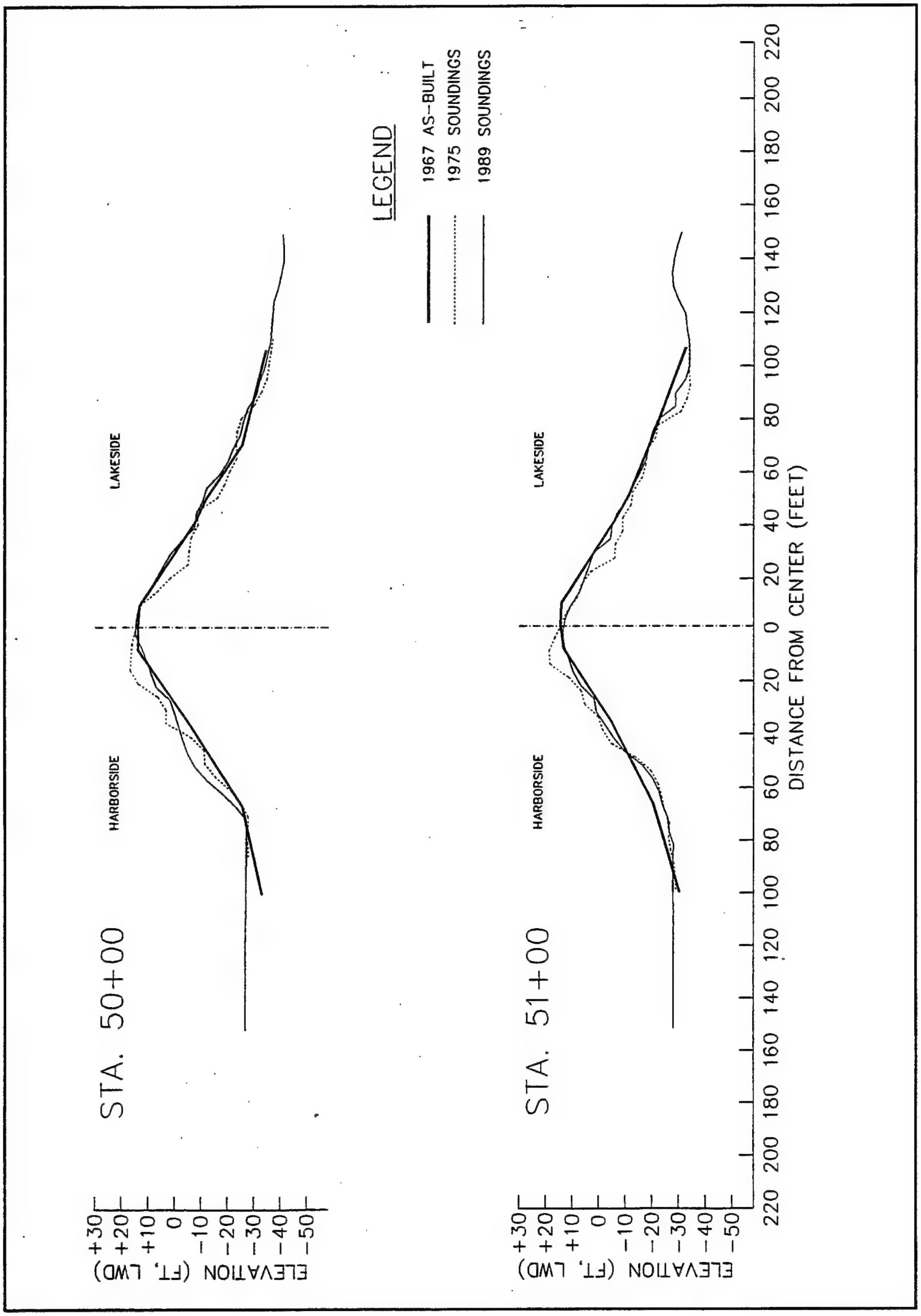
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ELEVATION (FT, LWD)
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ELEVATION (FT, LWD)
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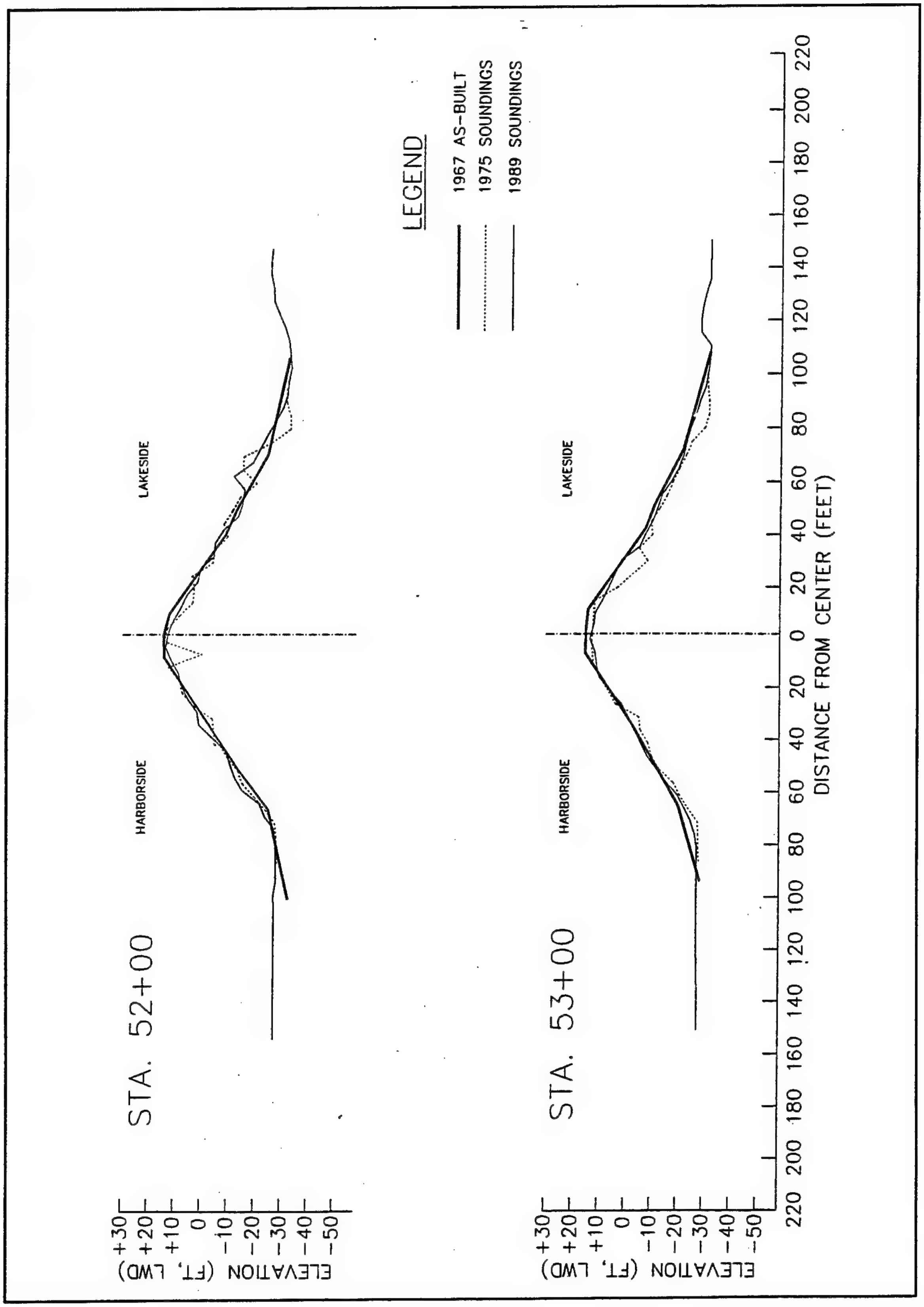

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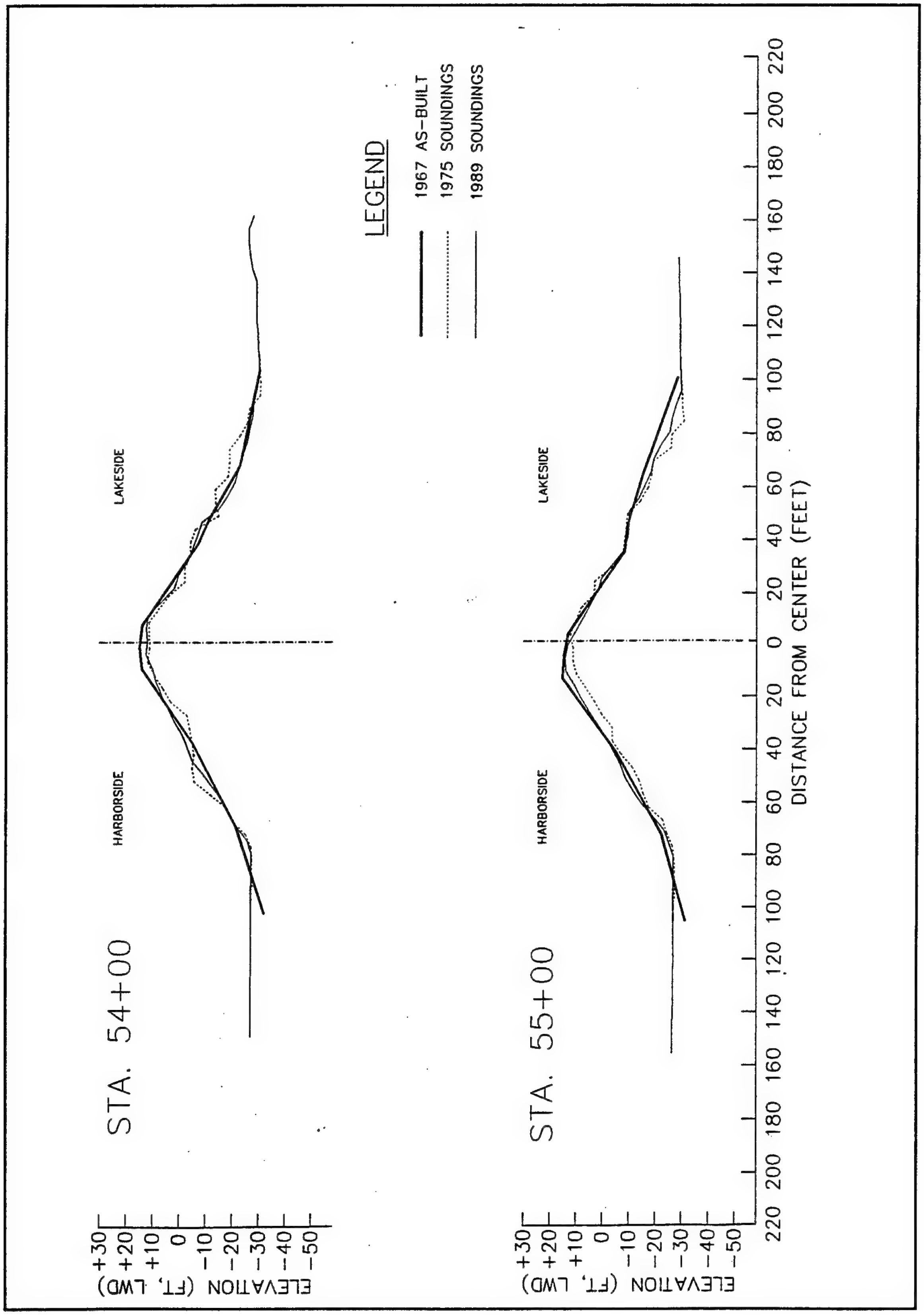
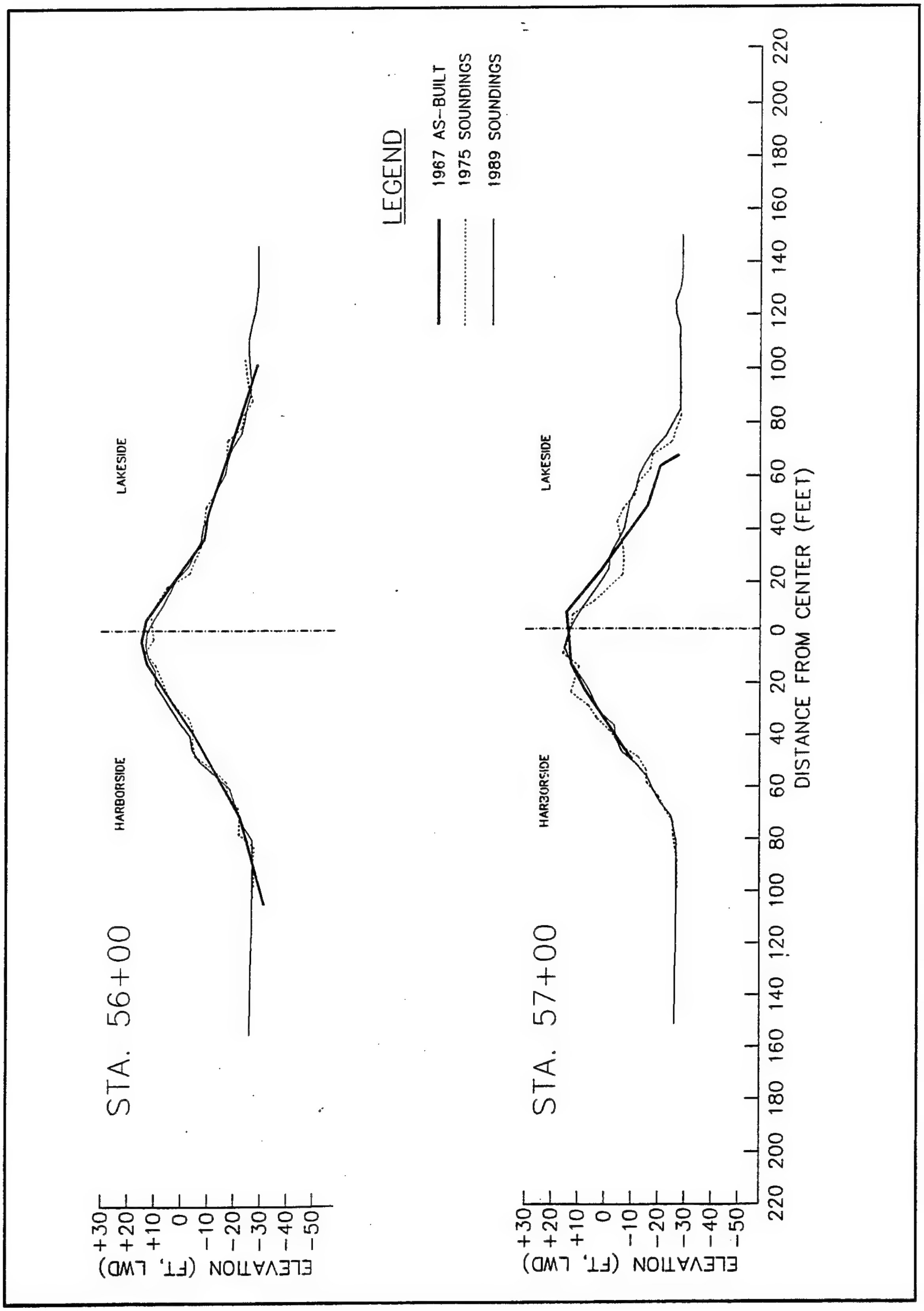
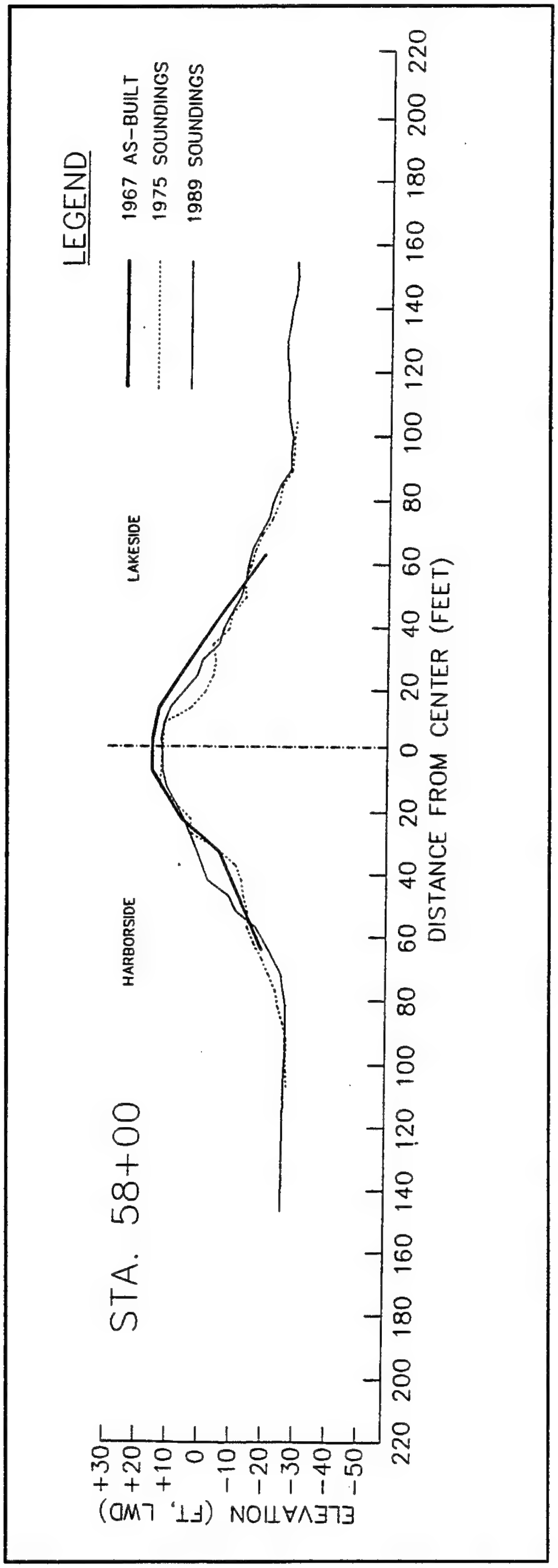
1967 AS-BUILT
1975 SOUNDINGS
1989 SOUNDINGS
ELEVATION (FT, LWD)
+30 +20 +10 0 -10 -20 -30 -40 -50
ELEVATION (FT, LWD)
+30 +20 +10 0 -10 -20 -30 -40 -50
220 200 180 160 140 120 100 80 60 40 20 0 20 40 60 80 100 120 140 160 180 200 220
DISTANCE FROM CENTER (FEET)


STA. 54+00
STA. 55+00
HARBORSIDE
LAKESIDE
LEGEND
1967 AS-BUILT
1975 SOUNDINGS
1989 SOUNDINGS
ELEVATION (FT, LWD)
+30
+20
+10
0
-10
-20
-30
-40
-50
DISTANCE FROM CENTER (FEET)
220 200 180 160 140 120 100 80 60 40 20 0 20 40 60 80 100 120 140 160 180 200 220


STA. 56+00
STA. 57+00
HARBORSIDE
LAKESIDE
LEGEND
1967 AS-BUILT
1975 SOUNDINGS
1989 SOUNDINGS
ELEVATION (FT, LWD)
+30
+20
+10
0
-10
-20
-30
-40
-50
DISTANCE FROM CENTER (FEET)
220 200 180 160 140 120 100 80 60 40 20 0 20 40 60 80 100 120 140 160 180 200 220


STA. 58+00
LEGEND
1967 AS-BUILT
1975 SOUNDINGS
1989 SOUNDINGS
HARBORSIDE
LAKESIDE
ELEVATION (FT, LWD)
+30 +20 +10 0 -10 -20 -30 -40 -50
220 200 180 160 140 120 100 80 60 40 20 0 20 40 60 80 100 120 140 160 180 200 220
DISTANCE FROM CENTER (FEET)

Appendix 5B
Horizontal Control History

DISCUSSION OF SURVEY, MONUMENTATION AND CONTROL HISTORY FOR BURNS HARBOR BREAKWATER

Comparison of as-built (design) centerline vs. 1975 structural centerline.

The 1975 Rip-Rap Sounding Survey was reviewed to determine the relevance of the **baseline** and structural centerline as shown on the 1975 Rip-Rap Survey. All spot elevations and soundings taken to generate the 1975 Rip-Rap Survey were done so with respect to the **baseline** as surveyed in the 1975. The **baseline**, as originally surveyed in 1975 and used for rip-rap sounding/vertical/horizontal control, was offset with respect to the structural centerline, also shown in the 1975 Rip-Rap Survey. The structure centerline is officially monumented with respect to the 1977 monumentation control map. However, the **baseline** (1975) was used as a basis for field control for the 1975 Rip-Rap Surveys and all subsequent repair surveys of the west arm and north arm of the breakwater.

The structural centerline of the breakwater, as shown in the 1975 Rip-Rap surveys, was intended to represent the as-built centerline. Upon careful examination of the project's as-built drawings (breakwater profiles), the as-built centerline was determined as being equivalent to the design centerline as presented in the project's contract drawings (1966). According to the 1975 Rip-Rap Survey and control map, the physical offset of the **baseline** with respect to the structural centerline were as follows:

* Station 0+00 to 4+00 the **baseline** varies 8 feet to 0 feet landward of centerline.

* Station 4+00 to 37+00 the **baseline** varies 0 feet to 10 feet lakeward of centerline.

* Station 37+00 to 46+00 the **baseline** varies 5 feet to 8 feet landward of centerline.

* Station 46+00 to steel cell the **baseline** is 5 feet lakeward of centerline.

If one were to rely exclusively upon the 1975 rip-rap surveys for supporting documentation, then the origin of the 1975 structural centerline was in doubt. This represented a major issue for the breakwater analysis in that the relation between the as-built centerline, 1975 survey **baseline**, and 1977 formal remonumented

centerline could not be directly related to coincide with one another. ED-GC needed to have these lines of survey/control related to one another in order to define the degree of reconfiguration, if any, the breakwater has underwent since its construction in 1967-1969.

Two hypotheses were given to explain the presence and significance of the 1975 structural centerline as shown on the rip-rap soundings.

(1) The 1975 orientation of the structural centerline, as shown on the 1975 rip-rap surveys, was incorrect and should have resembled the orientation as determined by a Coordinate-Geometry analysis performed by ED-DC (Dave Keil, 1990). This observation assumed that the structural centerline as presented in the 1975 rip-rap survey represented the design centerline (as-built centerline). The CO-GO analysis compared the 1975 structure centerline to the 1977 monumented centerline.

(2) If the structural centerline of the 1975 Rip-Rap Survey was assumed to only represent an apparent location for the breakwater crest, then it would be expected that the 1975 structural centerline would not have to match the as-built centerline given in state plane coordinates. This assumption would make the comparison of the 1976 breakwater analysis to the MCCP program analysis problematic.

Observation (1) turned out to be the most applicable for explaining the significance of the 1975 structure centerline. The following is an explanation of the origin for the 1975 centerline.

The 1975 structural centerline, as shown on the rip-rap soundings, was not documented from field observation, nor was it apparently documented by the Kewaunee Field office (POC Mr. Earl Neinas). Earl Neinas stated that the copies of the 1975 Rip-Rap Surveys in the Kewaunee office did not have the structural centerline noted on the drawings. Earl Neinas also stated that he was not aware of any such centerline notation with regard to the 1975 Rip-Rap Surveys and 1977 Control Monumentation. This is supported in that Mr. Neinas had conducted a 1976 in-office examination of the as-built design and survey baselines. The structural centerline, as noted in the Chicago District drawings was not a part of the 1975 survey line comparison, which Mr. Neinas had conducted.

The structural centerline was assumed to be incorporated into the 1975 Rip-Rap Survey drawings by the Chicago District office after the initial completion of the drawings. It is speculated that the structural centerline was incorporated into the 1975 Rip-Rap drawings through an (Chicago District) in-office positioning technique using non-surveyed information. The addition of the

structural centerline to the 1975 rip-rap survey "after-the fact" can be seen when the original linen/ink copies of the Rip-Rap Survey drawings are examined. The source information for the 1975 centerline was speculated to be the as-built (design) centerline.

The 1975 **baseline**, as shown on the 1975 Rip-Rap Survey, was intended to represent the (1966) design centerline. The 1975 **baseline** was developed in the Kewaunee office from information supplied by the Chicago District office. The **baseline** was surveyed at the field site by the Kewaunee office and was noted as being inconsistent with the design centerline (as-built centerline). The problem of survey irreplaceability of the design centerline was not due to field survey techniques, but was probably due to incorrect information transferred from the Chicago District to the Kewaunee field office.

The structural centerline, as shown on the Chicago District office 1975 Rip-Rap drawings, was determined to represent the as-built/design centerline as based on the location of northerly and easterly State plane coordinates.

Graphical presentation of the various centerline and **baseline** configurations is shown in figures 2 and 3. Below is given the relative distance of each breakwater segment at Burns Harbor w.r.t. individual centerline or **baseline** configurations.

	As-Built	1975		1977*
	Centerline	Baseline	Centerline	Centerline
BM 75-1 to R3	890	858.5	891	889.09
R3 to R4	3049	3144.3	3050	3058.15
R4 to BM 75-2	699	643.3	699	688.68
TOTAL	4638	4646	4640	4635

* As defined by Controls Map .

Conclusions:

The design centerline (1966) was adhered to fairly well during construction, but there is some variation when as-builts are compared to the 1975 and 1977 centerlines.

The 1975 **baseline** is not considered useful for crest location comparison purposes (re: cross-sections). The turning-angle location (in N-S) are not in agreement for the Design, 1975, and 1977 centerlines and turning-angle values.

Overall distances of the breakwater for each year's centerline and breakwater segment distance agree for the design (as-built),

1975, and 1977 centerlines. The 1975 rip-rap **baseline** is not in agreement with individual segment lengths of other survey configurations, but does agree in the overall aspect of breakwater length.

* The 1975 centerline is assumed to mimic the Design centerline.

* The 1977 centerline is assumed to represent the 1975 centerline with formal monumentation and control.

* Apparent changes in horizontal (and possibly vertical) control with respect to monumentation (bench marks 75-1 and 75-2 and interior turning points R-3 and R-4) of breakwater from as-built configuration to that of the 1975 and 1977 surveys:

  The 1977 control survey was an effort to reproduce in the field, the 1975 centerline configuration as drawn out in the office. Due to minor control error, induced by new monumentation, the two centerlines do not perfectly agree.

Other Factors Influencing Centerline Control

New monumentation was implemented at the ends of the breakwater at benchmarks 75-1 and 75-2 during a 1977 control activity. The endpoint monumentation along the breakwater's north arm appears vastly different in 1977 than the original as-built monumentation used for the 1975 rip-rap survey control.

It is unknown whether turning points (respective to locations R-3 and R-4) which were monumented in 1967-1968 were ever repeated in the 1977 control survey with respect to location and turning angle. This further complicates the assumption of coincidence of centerlines.

The four points of control along the north arm of Burns Harbor breakwater are a source of offset error which are reflected in the comparison between the 1975 (or as-built) and 1977 centerlines.

Comparison of the as-built (design) and 1975 centerlines is permissible, knowing that the representation of the as-built centerline by using the design centerline is more of a convention than reality.

Comparison of centerlines, surveyed later than 1975, to the as-built or the 1975 centerline can not be accurately done due to the uncertainty of monumentation correlation of dated surveys (1977 and later) to earlier surveys (1975 and earlier).

4

Subsequent surveys performed after 1977 along the breakwater's north arm continued to reference the old and out-dated as-built monumentation. This mis-monumentation affects the horizontal control and reliability of the reproductive surveys.

It is speculated that the orientation of 1977 monumented centerline to the 1975 structure centerline is what generated the offset which ED-DC submitted to ED-GC to permit the direct comparison of surveys based on the design centerline and surveys based on the 1975 **baseline**.

Comparison of the As-built Centerline and
Design centerline & Cross-sectional Alignment

Review and comparison of as-built (1968-69) and design (1967) cross-section/alignment indicated that as-built configurations of breakwater perfectly matched with the design configuration: both with respect to the structural centerline and crest width & orientation. This "perfect" match was evident for every cross-section, and gives rise to suspicion. Given the size of armor stone (i.e., internal tolerance) and the remoteness of construction (i.e., marine plant and lack of precise horizontal control), it must be assumed that complete agreement between the as-built configuration and design configuration can not be possible as portrayed in the project as-built specifications.

The structure centerline as presented in the as-built (cross-section documentation) survey can not be relied upon as representing the actual centerline of the breakwater structure upon construction. Therefore, all subsequent surveys which compare a relative **baseline** or centerline to the as-built centerline can not be assumed to have perfectly coincident centerlines with the true structure centerline; even if the subsequent survey centerline was based upon the exact location of the as-built centerline.

Verification of the 1976 Report Damage Assessment

The 1976 report, Review Report for the Performance of the Federal Breakwater at Burns Harbor , had assumed coincidence of 1975 structural centerline as compared to a true as-built centerline. This was checked in order to verify that no offset had been incorporated during the 1976 analysis/comparison of as-built centerline (cross-sections) and the 1975 structural centerline (cross-sections). Approximately 20% of the 1975 cross-sectional profiles had been shifted either to the lakeside or harborside, w.r.t. the as-built structure centerline in order to match the "foot-print" of the as-built profiles to the 1975 profiles.

Consequently, the maximum offset between the two profile sets was

4 feet, while the typical offset was 2 feet. Comparing sections which were "offset" in the 1976 report, the 2-foot "offset" did not significantly influence the overall damage trend which would have been represented the "offset" was ignored. Therefore the 1976 Damage Analysis does not appear to be influenced by unacceptable bias error due to "adjustments".

What was problematic for the prediction of breakwater response (damage) of the 1990 analysis conducted by Heidi L. Pfeiffer (Chicago District), was the assumption that the 1977 monumentation documented the location of the 1975 baseline.

The 1976 report, as mentioned above, relied upon the coincidence of the as-built centerline to that of the 1975 structural centerline to estimate relative change in the breakwater cross-section. Upon review of the 1967-1969 as-built cross-section, it was evident that a certain degree of "fit" was induced in order to "perfectly" match the as-built centerline to the construction design centerline. Therefore, it appeared uncertain as to where the actual as-built centerline was. Comparing subsequent survey structural centerlines to as-built centerlines does not guarantee the correct estimate of cross-sectional change (i.e. damage) sustained by the breakwater between 1967-1969 to the time frame (survey ref.) of interest. The following conclusions were inferred with regard to the above assessments:

(1) The 1967-1969 as-built centerlines may not be the true representation of actual structure centerline locations upon construction.

(2) Direct comparison between 1975 survey information of break water (re: structural centerline) and as-built centerline may not yield true estimates of damage sustained by the breakwater between 1967-1969 and 1975.

(3) Portions of the rubble mound breakwater may be built on unprepared lakebed if breakwater stone had not been placed in-line with prepared lake bottom.

(4) The 1976 Damage Analysis report may have been influenced by the erroneous assumption of the as-built (described previously) centerline representing the true initial position of the breakwater's centerline. However, there is no straight forward way to determine the degree of damage analysis error associated with the as-built assumption.

<u>Construction of the Burns Harbor Breakwater According to the Tolerances and Centerline as Directed in the Design Plan.</u>

Information supplied for the following discussion originated from conversations with Kewaunee Area Office (NCE) personnel who were

directly involved with the maintenance and repair of Burns Harbor breakwater during 1975-1980.
Dick Thibodeau and Darell Pederson (Kewaunee Area Office) stated that the breakwater could have been constructed outside of the alignment and centerline configuration as specified in the design plan.

The remoteness of construction and the method of constructing this type of structure may have influenced the control of the breakwater's alignment to a significant degree. For example, the length of each straight element of the breakwater's north arm is different when comparing the design (1966) centerline alignment and the monumented centerline alignment as surveyed in 1977. Turning angles and turning point locations (R3,R4) within the interior alignment of the breakwater vary according to year surveyed.

Earl Neinas stated that it would have been possible to have surveyed the breakwater in 1975, accurately enough as to reproduce the design centerline of the original breakwater plan. This could have been accomplished if the goal of the survey was to reproduce the original centerline in quality.

However, the requirement for such a survey was not mandated in 1975 due to the lack of need for absolute comparison of the overall structure from 1967 to 1975. Although the damage analysis preformed in 1976 tried to assess the overall breakwater change which occurred between 1967 and 1975, the 1975 surveys were not performed to allow for direct interpretation for this kind of analysis. Instead, the breakwater was surveyed for cross-sectional assessment (as compared to design template). The absolute (re)location of the 1967 centerline (1975 **baseline**) was not the primary goal of the 1975 rip-rap survey. The rip-rap survey was performed to only determine the relative configuration of the 1975 cross-sections.

Field Observations during Breakwater Repair
at Burns Harbor - April 85

Typically, the crest elevations along breakwater stations 0+00 to 4+00 were significantly lower than the rest of the breakwater.

The Chicago District geologist noted some fractured stone but not in the quantity to contribute to breakwater failure. Potential factors for influencing degradation were postulated as:

(1) lack of foundation preparation/adherence to prepared outlay;

(2) stone fracturing;

(3) armor stone rafting by ice encrustation on the crested area;

(4) leaching of core-stone through the breakwater Armor = 10-16 ton, secondary layer = 3-10 ton, bedding stone 1500-3000 lbs; and core stone = 1-50 lb.

(5) littoral or seiche induced current scouring sand fill, thus allowing the dike to settle near the harbor entrance or throughout the breakwater, where currents exist.

Comparisons of Larger armor stone used elsewhere in the Chicago Area and respective longevity.

Chicago Harbor (1900's) - 7 to 20 tons/stone

Milwaukee Harbor (1900's) - 10 to 20 tons/stone

These harbor locations are not subjected to as harsh wave conditions as encountered at Burns Harbor (due to fetch limitations). Yet, have incorporated larger armor stone sizes than used at the Burns Harbor breakwater.

Stone sizes and characteristics of recent rehabilitation measures on Chicago District breakwaters.

Calumet Harbor (1988) - 8 to 20 tons/stone, density = 165 lb/cft

Burns Harbor (1988) - 12 to 20 tons/stone, density = 165 lb/cft
- 10 to 16 tons/stone, density = 165 lb/cft

Recommendations based on field observation during the 1985 repair operations are as follows:

* Incorporate better/durable stone (denser and larger), the Bedford limestone originally used at the breakwater has a density of 145 lb/cft.

* Achieve better interlock in stone placement than what is shown in the breakwater. To date the breakwater had been maintained using a laid-up placement technique which would have given maximum interlock. The original construction technique was random placement, which provided little interlock. Some of the maintained sections have been disrupted since the 1975-80 maintenance program.

* Note that non-interlocked stone susceptible to ice rafting, heaving and sliding, and wave pressures and surging

8

Appendix 5C NCC Geotechnical Settlement Analysis (Reinvestigation)[1]

[1] Portions of information in the following NCC memorandums were taken from a 1991 draft report evaluating Burns Harbor breakwater settlement prepared by the WES Geotechnical Laboratory (GL) (see Chapter 4, this volume). The GL report (Chapter 4) has been revised since the memorandums were prepared in July and September 1993.

CENCC-ED-GT 29 July 1993

MEMORANDUM FOR RECORD

SUBJECT: Burns Waterway Harbor Breakwater, Indiana - Settlement Analysis

1. The Geotechnical Section has completed a settlement analysis of the subject breakwater. The analysis was undertaken in an attempt to identify a cause that would justify the stone losses which have occurred since construction of the breakwater. The recently completed "Burns Waterway Harbor, Indiana, Breakwater Major Rehabilitation Evaluation Report" states that nearly 145,000 tons of maintenance armor stone has been placed on the structure. Of this, approximately 69,000 tons cannot be accounted for.

2. The original Design Memorandum (DM) prepared for construction of the breakwater in 1966 discussed settlement. The DM states that settlements on the order of 2.5 feet at the centerline were possible if the existing foundation conditions were not modified. The report concludes that longitudinal distortions due to post-construction settlement of the breakwater were unlikely to have an adverse affect on its performance. State of the art concepts in 1966, however, did not permit realistic estimates of transverse distortions to be made. The report also recommends that, where the breakwater will rest on soft clay, a protective sand blanket, at least 3 feet thick, should be placed prior to construction.

3. The final design of the breakwater called for removal of soft clay up to 20 feet thick from the upper part of the breakwater foundation and replacement with sand to a specified depth.

4. In October 1991, the Geotechnical Laboratory of the U.S. Army Engineer Waterways Experiment Station (WES) drafted a report documenting their evaluation of the Burns Waterway Harbor breakwater. The purpose of the report was to determine if settlement of the breakwater structure could be a cause contributing to the reported unsatisfactory performance of the structure. The report concludes that it is unlikely that settlement has been a factor in the reported unsatisfactory performance of the breakwater.

5. The WES study addressed settlement for four conditions.

a. As-designed with clay removed and replaced with sand.

b. In-situ condition with no soft clay removal. This is the same case analyzed in the 1966 DM.

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c. Hypothetical case 1 in which the soft clay was removed, but 5 feet of soft clay lakebed was washed back into the excavation prior to sand placement.

d. Hypothetical case 2 is the same as 5c above except that 10 feet of lakebed was assumed to have washed back into the excavation prior to sand placement.

6. The WES report, however, does not address the issue of missing stone. The Chicago District's analysis was performed to try and find a correlation between breakwater settlement and the volume of missing stone.

7. Appendix A2 of the Burns Harbor Rehabilitation Report subdivided the breakwater into 8 segments. A soil profile was generated for each of these 8 segments based on the subsurface investigation performed in 1965. Based on the best available information from actual dredging during construction, the depth of clay removed and sand replaced were delineated on the profiles.

8. The soil boring information and consolidation data were applied over the following portions of the breakwater.

Boring	Stations
33	00+00 - 06+00
32	06+00 - 10+00
31	10+00 - 15+00
30	15+00 - 21+00
29	21+00 - 26+00
28	26+00 - 30+00
27	30+00 - 33+00
26-2	33+00 - 36+00
26	36+00 - 40+00
25	40+00 - 44+00
24	44+00 - 49+00
23	49+00 - 55+00
22	55+00 - 58+00

9. The average density of the structure was calculated. The calculations and the typical section used are attached.

10. The load under the centerline of the breakwater was calculated by multiplying the average height (55 feet) times the average density (51.7 pcf) resulting in a loading of 2850 psf.

11. The elevation of the lake bottom varies between 540 and 550 NGVD. Bedrock was assumed at elevation 460 NGVD. Thick soil strata were divided into layers averaging 20 feet thick. Stress changes in the different layers were determined using influence charts developed by Perloff (Foundation Engineering Handbook,

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1975, Fang and Winterkorn, pg 168). Settlements were calculated at both the centerline and toe of the breakwater.

12. Settlement of the breakwater was assumed to form a parabolic depression in the surface of the lakebed. A settlement volume was estimated by multiplying the area contained between the original lakebed surface and the parabolic settlement curve, by the distance representative of this settlement in accordance with paragraph 8 above.

13. The equivalent tons of stone which could fit into this volume was determined by multiplying the volume by a conversion factor of 1.37 tons per cubic yard.

14. The results indicate that approximately 57,900 tons of stone would have been lost through settlement as a result of the initial construction. Calculation summary sheets are attached. Using the settlement values presented in the 1991 WES report, about 51,000 tons of stone would have been lost.

15. Additional calculations were performed to determine the amount of additional settlement that would have occurred from placement of the maintenance stone since its construction. The amounts of stone placed in each segment are presented in table 3 of appendix A2 of the Burns Harbor Rehabilitation Evaluation Report. It was estimated that 70 percent of the maintenance stone was submerged. → ie underwater?

16. The results of this analysis suggest that an additional 2900 tons of stone would have been lost through settlement as a result of the placement of maintenance stone.

17. Our calculations estimate the total loss of stone at about 60,800 tons.

18. It should be noted that the settlement calculations, and in turn the volume estimates, are directly proportional to the compression index of the soil. In his book, Harr states that the coefficient of variation for the compression index for clays can be as high as 30 percent (Reliability-Based Design in Civil Engineering, 1987, Harr, pg 30). Corps manual EM 1110-1-1904 (1990), Settlement Analysis, states that settlement predictions are accurate to within 50 percent of actual settlements.

19. An estimate of the time rate of consolidation was also performed. Calculations assumed both one and two directional drainage.

20. The report, "Ground Water Resources of Northwestern Indiana, Bulletin No. 10 (1961)", prepared by the State of Indiana Department of Conservation, Division of Water Resources, states that glaciofluvial sand and some gravel underlie much of Lake

County and are the chief source of ground water in the unconsolidated rocks. Also, glacial deposits generally contain partings of sands and silts. For these reasons, we feel that the two directional drainage assumption is more realistic.

21. The escape of water during consolidation is a three dimensional problem. Therefore, consolidation theory provides an approximation at best. The real rate can only be determined by observation. Personal experiences suggest that the theory provides an upper limit.

22. Time rate of consolidation calculations are attached. The results indicate that it will take about 40 years to achieve 90 percent primary consolidation. This suggests that, to date, the breakwater has experienced an estimated 80 percent of its total anticipated settlement.

23. As stated in paragraph 20, it is our opinion that the structure has actually settled faster than the calculations indicate. What is important is that the calculations seem to confirm that the anticipated settlements have occurred over the same period of time, and at about the same rate as the stone placement.

<u>Conclusions</u>

24. Based on our analysis of the breakwater, we believe that there is adequate evidence to suggest that the amount of missing stone is related to the expected settlement of the breakwater.

John Fornek
JOHN T. FORNEK, P.E.
Geotechnical Engineer

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surcharge = 2850 psf, rev. 08/02/93
parabolic

SEGMENT 1 (B33)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			412.8	1567.5			strain=0.013	20	0.26			
	II	0	1	1438.5	1425.0	1.991	0.299	0.0995	18	0.54			
	III	0	1	3075.8	1140.0	1.371	0.137	0.0494	17	0.11			
	IV	0	1	4696.2	912.0	1.194	0.077	0.0494	20	0.06			
									sum	0.98			
TOE													
	I			412.8	0.0				20	0.00			
	II	0	1	1438.5	427.5	1.297	0.113	0.0995	18	0.20			
	III	0	1	3075.8	427.5	1.139	0.057	0.0494	17	0.06			
	IV	0	1	4696.2	458.0	1.097	0.040	0.0494	20	0.04			
									sum	0.29	93811	9478	4785

SEGMENT 2 (B31@ STA. 18+00)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I	0	1	597.3	1539.0	3.577	0.553	0.1484	22	1.81			
	II	0	1	2902.0	1262.5	1.429	0.155	0.1484	24	0.55			
	III	0	1	3718.0	997.5	1.268	0.103	0.0494	18	0.09			
	IV	0	1	4867.2	912.0	1.187	0.075	0.0494	18	0.06			
									sum	2.51			
TOE													
	I	0	1	597.3	256.5	1.429	0.155	0.1484	22	0.51			
	II	0	1	2902.0	427.5	1.143	0.058	0.1484	24	0.21			
	III	0	1	3718.0	427.5	1.115	0.047	0.0494	18	0.04			
	IV	0	1	4867.2	458.0	1.094	0.039	0.0494	18	0.03			
									sum	0.79	200265	7417	10181

SEGMENT 2 (B32)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			550.8	1567.5			strain=0.012	16	0.19			
	II	0	1	1679.0	1425.0	1.849	0.267	0.0995	15	0.40			
	III	0	1	2906.8	1140.0	1.392	0.144	0.0494	25	0.18			
	IV	0	1	4596.8	940.5	1.205	0.081	0.0494	24	0.10			
									sum	0.87			
TOE													
	I			550.8	0.0				16	0.00			
	II	0	1	1679.0	427.5	1.255	0.099	0.0995	15	0.15			
	III	0	1	2906.8	427.5	1.147	0.060	0.0494	25	0.07			
	IV	0	1	4596.8	456.0	1.099	0.041	0.0494	24	0.05			
									sum	0.27	55239	2048	2903

SEGMENT 3 (B30)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I	0	1	362.0	1567.5	5.330	0.727	0.1136	10	0.63			
	II			845.0	1536.0			strain=0.01	5	0.06			
	III	0	1	2172.0	1339.5	1.617	0.208	0.1136	30	0.71			
	IV	0	1	3549.0	1083.0	1.305	0.116	0.0494	35	0.20			
TOE									sum	1.79			
	I	0	1	362.0	0.0			0.1136	10	0.00			
	II			845.0	256.5			strain=0.002	5	0.01			
	III	0	1	2172.0	427.5	1.197	0.078	0.1136	30	0.27			
	IV	0	1	3549.0	427.5	1.120	0.049	0.0494	35	0.09			
									sum	0.36	163270	6047	8294

SEGMENT 4 (B29)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			280.4	1567.5			strain=0.014	8	0.11			
	II			846.0	1536.0			strain=0.011	8	0.09			
	III	0	1	1684.8	1339.5	1.795	0.254	0.0959	24	0.58			
	IV	0	1	3278.6	1083.0	1.330	0.124	0.0494	21	0.13			
	V	0	1	4608.2	912.0	1.194	0.077	0.0494	21	0.06			
TOE									sum	1.00			
	I			280.4	0.0				8	0.00			
	II			846.0	256.5			strain=0.002	8	0.02			
	III	0	1	1684.8	427.5	1.254	0.098	0.0959	24	0.23			
	IV	0	1	3278.6	427.5	1.130	0.053	0.0494	21	0.06			
	V	0	1	4608.2	456.0	1.097	0.040	0.0494	21	0.04			
									sum	0.34	80798	2992	4100

SEGMENT 4 (B28)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			508.2	1567.5			strain=0.012	14	0.16			
	II	0	1	2086.4	1530.5	1.636	0.214	0.0959	33	0.66			
	III	0	1	3751.8	907.5	1.266	0.102	0.0494	17	0.09			
	IV	0	1	4934.8	812.0	1.165	0.074	0.0494	18	0.07			
TOE									sum	1.01			
	I			508.2	0.0				14	0.00			
	II	0	1	2086.4	427.5	1.204	0.081	0.0959	33	0.25			
	III	0	1	3751.8	427.5	1.114	0.047	0.0494	17	0.04			
	IV	0	1	4934.8	456.0	1.092	0.036	0.0494	18	0.03			
									sum	0.33	64552	2391	3275

SEGMENT 6 (B27)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			624.6	1567.5			strain=0.012	16	0.21			
	II	0	1	1283.0	1530.0	2.200	0.342	0.0959	6	0.20			
	III	0	1	2630.4	1330.5	1.506	0.178	0.0494	30	0.26			
	IV	0	1	4664.4	1063.0	1.232	0.091	0.0494	30	0.13			
									sum	0.81			
TOE													
	I			624.6	0.0				16	0.00			
	II	0	1	1283.0	256.5	1.200	0.079	0.0959	6	0.06			
	III	0	1	2630.4	427.5	1.162	0.065	0.0494	30	0.10			
	IV	0	1	4664.4	427.5	1.092	0.038	0.0494	30	0.06			
									sum	0.20	37613	1393	1909

SEGMENT 6 (B26-2)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			555.2	1567.5			strain=0.012	16	0.20			
	II	0	1	1723.8	1425.0	1.827	0.262	0.0494	19	0.25			
	III	0	1	3008.2	1140.0	1.379	0.140	0.0494	19	0.13			
	IV	0	1	4596.8	940.5	1.205	0.081	0.0494	28	0.11			
									sum	0.69			
TOE													
	I			555.2	0.0				16	0.00			
	II	0	1	1723.8	427.5	1.248	0.096	0.0494	19	0.09			
	III	0	1	3008.2	427.5	1.142	0.058	0.0494	19	0.05			
	IV	0	1	4596.8	459.0	1.099	0.041	0.0494	28	0.06			
									sum	0.20	32480	1203	1646

SEGMENT 6 (B26)

CENTER	LAYER	Co	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			336.6	1567.5			strain=0.014	9	0.12			
	II	0	1	1278.6	1536.0	2.208	0.344	0.1803	5	0.31			
	III	0	1	1994.2	1330.5	1.672	0.223	0.0494	31	0.34			
	IV	0	1	4225.0	969.0	1.229	0.090	0.0494	36	0.15			
									sum	0.93			
TOE													
	I			336.6	0.0				9	0.00			
	II	0	1	1278.6	256.5	1.201	0.080	0.1803	5	0.07			
	III	0	1	1994.2	427.5	1.214	0.084	0.0494	31	0.13			
	IV	0	1	4225.0	456.0	1.108	0.045	0.0494	36	0.06			
									sum	0.26	59003	2185	2994

SEGMENT 6 (B25)

CENTER	LAYER	Co	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			448.8	1567.5			strain=0.013	12	0.15			
	II	0	1	1498.5	1536.0	2.027	0.307	0.1803	3	0.17			
	III	0	1	1218.8	1425.0	2.171	0.337	0.1045	9	0.32			
	IV	0	1	2332.2	1282.5	1.550	0.190	0.0494	21	0.20			
	V	0	1	3751.8	897.5	1.298	0.102	0.0494	21	0.11			
	VI	0	1	4934.8	969.0	1.196	0.078	0.0494	14	0.05			
									sum	0.90			
TOE													
	I			448.8	0.0				12	0.00			
	II	0	1	1498.5	256.5	1.171	0.068	0.1803	3	0.04			
	III	0	1	1218.8	427.5	1.351	0.131	0.1045	9	0.12			
	IV	0	1	2332.2	427.5	1.183	0.073	0.0494	21	0.08			
	V	0	1	3751.8	427.5	1.114	0.047	0.0494	21	0.05			
	VI	0	1	4934.8	456.0	1.092	0.038	0.0494	14	0.03			
									sum	0.31	83481	2351	3221

SEGMENT 6 (B24)

CENTER	LAYER	Co	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			497.7	1567.5			strain=0.013	14	0.18			
	II	0	1	1497.6	1425.0	1.952	0.290	0.1045	20	0.61			
	III			2572.6	1197.0			strain=0.002	8	0.01			
	IV	0	1	3120.0	1063.0	1.347	0.129	0.1045	18	0.22			
	V	0	1	4664.4	940.5	1.202	0.080	0.0494	22	0.09			
									sum	1.10			
TOE													
	I			497.7	0.0				14	0.00			
	II	0	1	1497.6	427.5	1.285	0.109	0.1045	20	0.23			
	III			2572.6	427.5			strain=0.0007	8	0.01			
	IV	0	1	3120.0	427.5	1.137	0.056	0.1045	18	0.09			
	V	0	1	4664.4	456.0	1.098	0.041	0.0494	22	0.04			
									sum	0.37	96718	3288	4501

SEGMENT 7 (B23)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			417.5	1567.5			strain=0.004	5	0.03			
	II	0	1	540.4	1539.0	3.848	0.585	0.1500	6	0.53			
	III	0	1	1684.8	1339.5	1.795	0.254	0.1045	20	0.53			
	IV	0	1	3312.4	1083.0	1.327	0.123	0.0494	24	0.15			
	V	0	1	4799.6	912.0	1.190	0.079	0.0494	20	0.07			
									sum	1.30			
TOE													
	I			417.5	0.0				5	0.00			
	II	0	1	540.4	256.5	1.475	0.169	0.1500	6	0.15			
	III	0	1	1684.8	427.5	1.254	0.098	0.1045	20	0.21			
	IV	0	1	3312.4	427.5	1.129	0.053	0.0494	24	0.06			
	V	0	1	4799.6	456.0	1.095	0.039	0.0494	20	0.04			
									sum	0.48	126848	4698	6436

SEGMENT 8 (B22)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	Tons
	I			238.8	1710.0			strain=0.005	5	0.02			
	II	0	1	306.6	1567.5	6.076	0.784	0.1303	6	0.61			
	III	0	1	661.2	1530.0	3.259	0.513	0.1045	4	0.21			
	IV	0	1	1250.6	1482.0	2.185	0.339	0.0494	7	0.12			
	V	0	1	1545.8	1339.5	1.867	0.271	0.1045	15	0.42			
	VI	0	1	3447.6	1083.0	1.314	0.119	0.0494	28	0.16			
	VII	0	1	6070.0	956.0	1.169	0.068	0.0494	20	0.07			
									sum	1.83			
TOE													
	I			238.8	0.0				5	0.00			
	II	0	1	306.6	0.0			0.1303	6	0.00			
	III	0	1	661.2	256.5	1.377	0.139	0.1045	4	0.06			
	IV	0	1	1250.6	296.0	1.226	0.089	0.0494	7	0.03			
	V	0	1	1545.8	427.5	1.277	0.106	0.1045	15	0.17			
	VI	0	1	3447.6	427.5	1.124	0.051	0.0494	28	0.07			
	VII	0	1	6070.0	484.5	1.098	0.040	0.0494	20	0.04			
									sum	0.38	74818	2771	5796

TOTAL YARDAGE= 42259

TOTAL TONNAGE= 57804 57804

surcharge = 2850 psf, rev. 08/02/93
w/added stone, parabola

SEGMENT 1 (B33)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			1960.3	210.5			strain=0.013	20	0.03			
	II	0	1	2583.5	191.4	1.067	0.028	0.0985	18	0.05			
	III	0	1	4215.9	153.1	1.036	0.015	0.0494	17	0.01			
	IV	0	1	5610.2	122.5	1.022	0.009	0.0494	20	0.01			
									sum	0.10			
TOE													
	I			412.8	0.0				20	0.00			
	II	0	1	1896.0	57.4	1.031	0.013	0.0985	18	0.02			
	III	0	1	3503.3	57.4	1.016	0.007	0.0494	17	0.01			
	IV	0	1	5154.2	61.2	1.012	0.005	0.0494	20	0.01			
									sum	0.03	11107	411	594

SEGMENT 2 (B31@ STA. 19+00)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I	0	1	2136.3	96.0	1.046	0.019	0.1484	22	0.06			
	II	0	1	4274.5	81.7	1.019	0.008	0.1484	24	0.03			
	III	0	1	4715.5	63.5	1.013	0.006	0.0494	16	0.01			
	IV	0	1	5779.2	58.1	1.010	0.004	0.0494	16	0.00			
									sum	0.10			
TOE													
	I	0	1	853.8	16.3	1.019	0.008	0.1484	22	0.03			
	II	0	1	3419.5	27.2	1.008	0.003	0.1484	24	0.01			
	III	0	1	4145.5	27.2	1.007	0.003	0.0494	16	0.00			
	IV	0	1	5323.2	29.0	1.005	0.002	0.0494	16	0.00			
									sum	0.04	9720	360	493

SEGMENT 2 (B32)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			2136.3	99.9			strain=0.012	16	0.01			
	II	0	1	3104.0	90.9	1.029	0.013	0.0985	15	0.02			
	III	0	1	4046.5	72.6	1.018	0.008	0.0494	25	0.01			
	IV	0	1	5537.3	59.8	1.011	0.005	0.0494	24	0.01			
									sum	0.04			
TOE													
	I			566.8	0.0				16	0.00			
	II	0	1	2106.5	27.2	1.013	0.006	0.0985	15	0.01			
	III	0	1	427.5	27.2	1.064	0.027	0.0494	25	0.03			
	IV	0	1	5062.8	29.0	1.006	0.002	0.0494	24	0.00			
									sum	0.04	3578	133	182

SEGMENT 3 (S30)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I	0	1	1929.5	89.5	1.046	0.020	0.1136	10	0.02			
	II			2364.0	87.9			strain=0.01	5	0.00			
	III	0	1	3511.5	78.5	1.022	0.009	0.1136	30	0.03			
	IV	0	1	4632.0	61.9	1.013	0.006	0.0494	35	0.01			
TOE									sum	0.07			
	I	0	1	362.0	0.0			0.1136	10	0.00			
	II			1101.5	14.7			strain=0.002	5	0.00			
	III	0	1	2589.5	24.4	1.009	0.004	0.1136	30	0.01			
	IV	0	1	3978.5	24.4	1.006	0.003	0.0494	35	0.00			
									sum	0.02	7510	278	281

SEGMENT 4 (S29)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			1967.9	125.9			strain=0.014	8	0.01			
	II			2185.0	123.9			strain=0.011	8	0.01			
	III	0	1	3024.3	107.8	1.036	0.015	0.0659	24	0.03			
	IV	0	1	4361.8	87.0	1.020	0.009	0.0494	21	0.01			
	V	0	1	5610.2	73.3	1.013	0.006	0.0494	21	0.01			
TOE									sum	0.06			
	I			280.4	0.0				8	0.00			
	II			902.6	20.6			strain=0.002	8	0.00			
	III	0	1	2112.3	34.3	1.016	0.007	0.0659	24	0.02			
	IV	0	1	3708.1	34.3	1.009	0.004	0.0494	21	0.00			
	V	0	1	5154.2	36.8	1.007	0.003	0.0494	21	0.00			
									sum	0.02	5981	218	299

SEGMENT 4 (S26)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			2075.7	125.9			strain=0.012	14	0.01			
	II	0	1	3437.9	107.8	1.031	0.013	0.0659	33	0.04			
	III	0	1	4748.3	90.1	1.017	0.007	0.0494	17	0.01			
	IV	0	1	5848.8	73.3	1.013	0.006	0.0494	18	0.00			
TOE									sum	0.06			
	I			508.2	0.0				14	0.00			
	II	0	1	2525.9	34.3	1.014	0.006	0.0659	33	0.02			
	III	0	1	4179.3	34.3	1.008	0.004	0.0494	17	0.00			
	IV	0	1	5390.6	36.8	1.007	0.003	0.0494	18	0.00			
									sum	0.02	4842	179	248

SEGMENT 5 (B27)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			2182.1	82.1			strain=0.012	18	0.01			
	II	0	1	3822.0	81.0	1.022	0.009	0.0869	8	0.01			
	III	0	1	3875.9	53.1	1.013	0.006	0.0494	30	0.01			
	IV	0	1	5747.4	42.9	1.007	0.003	0.0494	30	0.00			
									sum	0.03			
TOE													
	I			924.6	0.0				18	0.00			
	II	0	1	1530.5	10.2	1.007	0.003	0.0869	8	0.00			
	III	0	1	3069.9	19.9	1.006	0.002	0.0494	30	0.00			
	IV	0	1	5091.9	18.8	1.003	0.001	0.0494	30	0.00			
									sum	0.01	1415	52	72

SEGMENT 5 (B26-2)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			2122.7	62.1			strain=0.018	18	0.01			
	II	0	1	3148.8	66.5	1.018	0.008	0.0494	18	0.01			
	III	0	1	4148.2	45.2	1.011	0.005	0.0494	18	0.00			
	IV	0	1	5537.3	37.3	1.007	0.003	0.0494	28	0.00			
									sum	0.02			
TOE													
	I			558.2	0.0				18	0.00			
	II	0	1	2151.3	19.9	1.009	0.003	0.0494	18	0.00			
	III	0	1	3436.7	18.9	1.005	0.002	0.0494	18	0.00			
	IV	0	1	5052.8	19.1	1.004	0.002	0.0494	28	0.00			
									sum	0.01	1221	45	62

SEGMENT 6 (B26)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			1804.1	71.5			strain=-0.014	9	0.00			
	II	0	1	2616.5	70.2	1.026	0.011	0.1603	5	0.01			
	III	0	1	3333.7	61.1	1.016	0.006	0.0494	31	0.01			
	IV	0	1	5194.0	44.2	1.008	0.004	0.0494	35	0.01			
TOE									sum	0.03			
	I			338.8	0.0				9	0.00			
	II	0	1	1833.0	11.7	1.006	0.003	0.1603	5	0.00			
	III	0	1	2421.7	19.6	1.006	0.003	0.0494	31	0.01			
	IV	0	1	4061.0	20.8	1.004	0.002	0.0494	35	0.00			
									sum	0.01	2433	90	123

SEGMENT 6 (B25)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			3016.3	71.5			strain=-0.013	12	0.01			
	II	0	1	3037.5	70.2	1.023	0.010	0.1603	3	0.01			
	III	0	1	3641.6	66.6	1.026	0.011	0.1045	8	0.01			
	IV	0	1	3914.7	59.5	1.016	0.007	0.0494	21	0.01			
	V	0	1	4749.3	46.6	1.010	0.004	0.0494	21	0.00			
	VI	0	1	5603.8	44.2	1.007	0.003	0.0494	14	0.00			
TOE									sum	0.03			
	I			446.8	0.0				12	0.00			
	II	0	1	1755.0	11.7	1.007	0.003	0.1603	3	0.00			
	III	0	1	1944.3	19.6	1.012	0.005	0.1045	8	0.00			
	IV	0	1	2756.7	19.6	1.007	0.003	0.0494	21	0.00			
	V	0	1	4179.3	19.5	1.005	0.002	0.0494	21	0.00			
	VI	0	1	5380.6	20.8	1.004	0.002	0.0494	14	0.00			
									sum	0.01	2606	96	132

SEGMENT 6 (B24)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			2095.2	71.5			strain=-0.013	14	0.01			
	II	0	1	2922.6	85.0	1.022	0.010	0.1045	20	0.02			
	III			3760.8	54.6			strain=-0.002	8	0.00			
	IV	0	1	4203.0	48.4	1.012	0.005	0.1045	16	0.01			
	V	0	1	5604.9	42.9	1.008	0.003	0.0494	22	0.00			
TOE									sum	0.04			
	I			497.7	0.0				14	0.00			
	II	0	1	1925.1	19.5	1.010	0.004	0.1045	20	0.01			
	III			3000.1	19.5			strain=-0.0007	8	0.00			
	IV	0	1	3647.5	19.5	1.005	0.002	0.1045	16	0.00			
	V	0	1	5130.4	20.8	1.004	0.002	0.0494	22	0.00			
									sum	0.02	3676	136	187

SEGMENT 7 (S23)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			1966.0	42.6			strain=0.004	6	0.00			
	II	0	1	2079.4	42.0	1.020	0.008	0.1500	6	0.01			
	III	0	1	3024.3	36.6	1.012	0.005	0.1045	20	0.01			
	IV	0	1	4396.4	29.6	1.007	0.003	0.0484	24	0.00			
	V	0	1	5711.6	24.9	1.004	0.002	0.0484	20	0.00			
									sum	0.02			
TOE													
	I			417.5	0.0				6	0.00			
	II	0	1	796.9	7.0	1.009	0.004	0.1500	6	0.00			
	III	0	1	2112.3	11.7	1.005	0.002	0.1045	20	0.01			
	IV	0	1	3730.6	11.7	1.003	0.001	0.0484	24	0.00			
	V	0	1	5255.6	12.5	1.002	0.001	0.0484	20	0.00			
									sum	0.01	2830	105	144

SEGMENT 8 (S22)

CENTER	LAYER	Cc	eo	Po	DEL P	Po+DEL P/Po	log(Po+DEL P/Po)	Cc/1+eo	H	DEL H	V (cf)	V (cy)	TONS
	I			1948.8	0.0			strain=0.005	5	0.00			
	II	0	1	1878.3	0.0	1.000	0.000	0.1303	6	0.00			
	III	0	1	2220.2	0.0	1.000	0.000	0.1045	4	0.00			
	IV	0	1	2732.6	0.0	1.000	0.000	0.0484	7	0.00			
	V	0	1	2886.3	0.0	1.000	0.000	0.1045	15	0.00			
	VI	0	1	4530.6	0.0	1.000	0.000	0.0484	28	0.00			
	VII	0	1	5825.0	0.0	1.000	0.000	0.0484	20	0.00			
									sum	0.00			
TOE													
	I			238.5	0.0				5	0.00			
	II	0	1	308.8	0.0			0.1303	6	0.00			
	III	0	1	837.7	0.0	1.000	0.000	0.1045	4	0.00			
	IV	0	1	1535.8	0.0	1.000	0.000	0.0484	7	0.00			
	V	0	1	1973.3	0.0	1.000	0.000	0.1045	15	0.00			
	VI	0	1	3675.1	0.0	1.000	0.000	0.0484	28	0.00			
	VII	0	1	5564.5	0.0	1.000	0.000	0.0484	20	0.00			
									sum	0.00	0	0	0

TOTAL YARDAGE=	2104	
TOTAL TONNAGE=	2863	2863

SHEET 1 OF 2

COMP. BY JF DATE 28 JULY 93

US Army Corps of Engineers
Chicago District

CHKD. BY [redacted]

PROJECT Burns [redacted] STRUCTURE TIME RATE SETTLEMENT

FROM LAMBE & WHITMAN

ASSUMED $C_v = 1 \times 10^{-3}$ cm²/sec = 0.09 FT²/DAY

$t = T_v \frac{H^2}{C_v}$ T_v USING CURVE C_1

U(%)	T_v	t (yrs) H=40'	t (yrs) H=80'
20	0.03	1.5	5.8
40	0.13	6.3	25.3
60	0.29	14.1	56.5
80	0.57	27.8	111.1
90	0.85	41.4	165.6

[redacted]

[redacted]

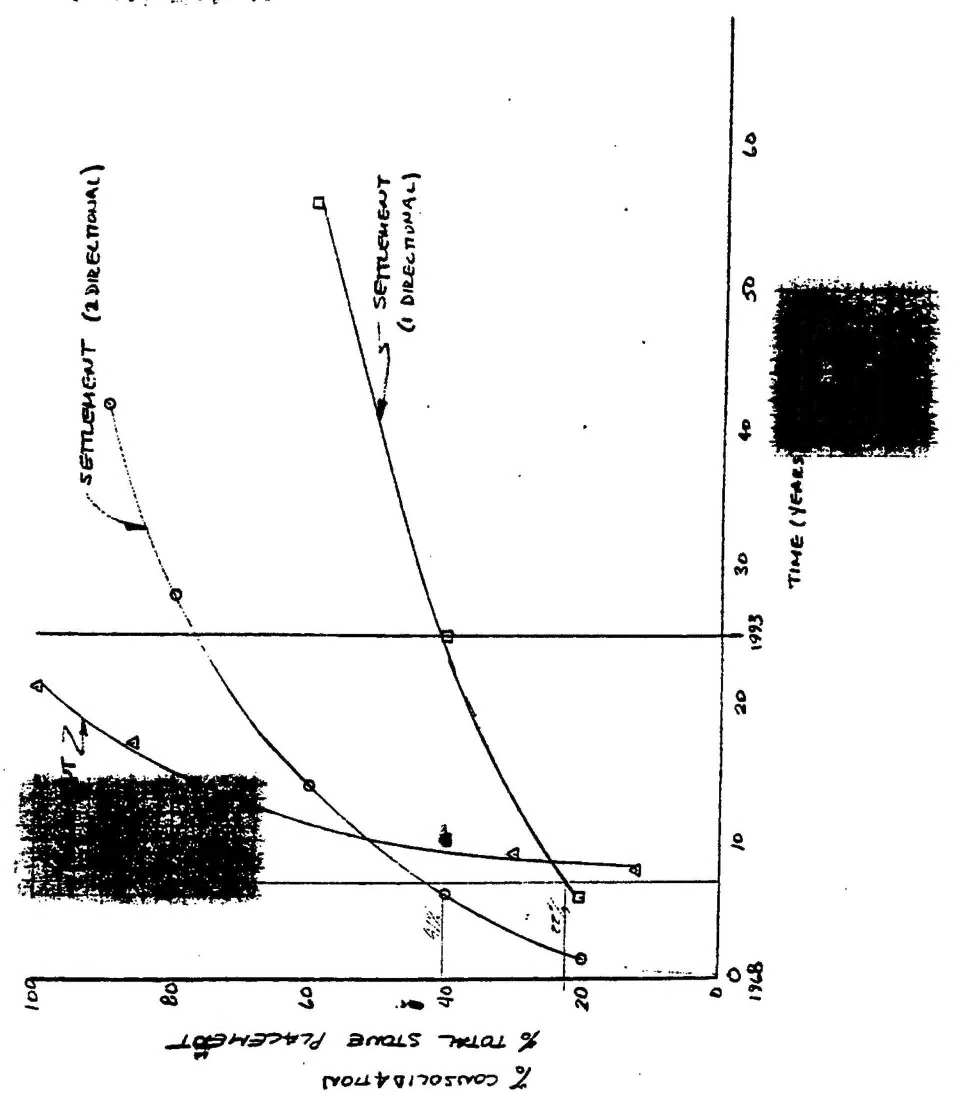
NCC FORM 1255
(1 OCT 92)

US Army Corps of Engineers
Chicago District

COMP. BY JF DATE 28 July 93 SHEET 2 OF 2

CHKD. BY [redacted]

PROJECT BURN[redacted]TER STRUCTURE TIME RATE SETTLEMENT



CENCC-ED-GT 9 September 1993

MEMORANDUM FOR RECORD

SUBJECT: Burns Harbor Breakwater Rehabilitation Settlement Calculations

1. References:

a. Memorandum for Record dated 29 July 1993 from CENCC-ED-GT; Subject: Burns Waterway Harbor Breakwater, Indiana - Settlement Analysis.

b. Sverdrup & Parcel and Associates construction progress reports, No. 4 (30 June 1966) through No. 16 (30 June 1968)

c. Letter dated 8 November 1967 from Peter Kiewit Sons' Company, Contractors to Sverdrup & Parcel and Associates (SPA); Subject: Dredging of Soft Clay Burns Waterway Harbor

d. Letter dated 20 November 1967 from SPA to Peter Kiewit; Subject: Dredging of Soft Clay Burns Waterway Harbor

e. Burns Waterway Harbor Evaluation of Settlement of Breakwater, Final Draft Report, 28 October 1991 prepared by John Anderson of W.E.S.

2. The Geotechnical Section has reviewed the soil profiles, settlement calculations, and other construction documents in an attempt to provide an explanation for the error between predicted and actual damages along the west reach of the subject breakwater. Reference 1b reports that the removal of soft, lake bed clay between stations 43+00 and 60+00 occurred during October and November 1967. Construction operations were suspended for the winter on 25 November 1967. Construction resumed and sand backfill was placed during April and June 1968. To the best of our knowledge, this is the only major shut-down that occurred during construction of the project.

3. Reference 1c discusses plans for sampling of clay beneath the sand blanket in the vicinity of station 57+50. The letter states that, if the clay is found to be satisfactory, the trench will be backfilled immediately in an effort to protect the lake bottom from erosion during the winter storm.

4. Reference 1d states that..."the material immediately below the sand blanket is Lake Clay, and, therefore, unsuitable for the breakwater foundation." The letter goes on to state that..."[a]ctual yardage will be computed during the winter shut-down", confirming the plan to suspend construction for the

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winter.

5. This area of Lake Michigan possesses a great deal of wave energy. It is possible for large quantities of lake bed sediments to be moved by storms during the winter season.

6. It is conceivable that the trench, which reference 1b indicates was not backfilled for nearly five months, may have become partially, or totally, filled with soft lake sediments during the winter of 1967. Construction records indicate that 7 to 16 feet of soft clay was initially removed during construction of the west leg with most of the excavation ranging between 10 and 13 feet.

7. Table 2 of reference 1e indicates that the re-deposition of 5 to 10 feet of soft clay in the trench could result in additional settlement of 0.4 to 0.6 feet, respectively. This is based on soil conditions in the vicinity of station 47+00.

8. The W.E.S. analysis estimated soil properties by a procedure developed by Sneathen et al. from work on remolded clays from underwater sediments. Based on this method, the compression index and void ratio were estimated at 0.44 and 1.04, respectively. Although W.E.S. states that they made a conservative selection for the soil parameters, our review of figure 16 in reference 1e shows that values for the compression index and void ratio may reasonably be as high as 0.7 and 1.15. These values would result in a 50 percent increase in anticipated settlement due to deposition of soft clay in the trench.

9. Reference 1e is correct in stating that..."the consistency of such material is difficult to represent in single form. Probably it would be a mixture of lumped chunks and scattered small particles of clay." "...Estimation of the consolidation parameters for clay which underwent the process of dredging, dumping, and uncertain mode of redeposition, is difficult."

10. The attached figure summarizes theoretical and actual damages for the breakwater. Based on the above discussion, the theoretical predictions for cross sectional area changes in segments 7 and 8 could increase between 62 and 140 square feet.

JOHN T. FORNEK, P.E.
Geotechnical Engineer

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REPORT DOCUMENTATION PAGE

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13. ABSTRACT *(Maximum 200 words)*

In 1984, Burns Harbor was nominated for inclusion in the Monitoring Completed Navigation Projects Program sponsored by Headquarters, U.S. Army Corps of Engineers. Burns Harbor was approved for monitoring in 1985 because it met both generic and site-specific selection criteria. This report contains independently prepared chapters with detailed descriptions of five major elements of the overall study at Burns Harbor which are: project history, analysis of wave measurements, extremal analysis of hindcast and measured wave data, evaluation of breakwater settlement, and a structural stability analysis.

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Wave reflection

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